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STATE OF THE ART REVIEW
SEPTIC TANK - DISPOSAL FIELD SYSTEMS

State of the Art Review

Septic Tank-Disposal Field Systems

Prepared by:

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Prepared for:

Alberta Environment
Municipal Engineering Branch
Standards and Approvals Division

August, 1985

Acknowledgements

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10 April 1985
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Dear Sir:

Reference: State-of-the-Art Review
Septic Tank - Disposal Field Systems

Attached please find our Final Report for the above study. We have appreciated the opportunity to participate in this most interesting project. Material presented in this document will be very useful in the development of design standards for the Province of Alberta.

Thank you for your interest and enthusiasm throughout this project. We look forward to our further participation with you.

Yours very truly,

STANLEY ASSOCIATES ENGINEERING LTD.

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EK/llg

Attachment

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SECTION 1

INTRODUCTION

1.1 HISTORICAL EVOLUTION OF SEPTIC TANK SYSTEMS

Today's septic tank is an outgrowth of the cesspool and the sedimentation tank. Devices which might be described as septic tanks are known to have existed as far back as 2000 B.C. Cotteral and Norris (1969) reported that the first system, similar to the septic tank as we know it today, was patented in France in 1881 by M. Mouras. It was called the Mouras Automatic Scavenger and was described as a "mysterious contrivance consisting of a vault hermetically closed by a hydraulic seal. By a mysterious operation, and one which reveals an entirely novel principle, it rapidly transforms all the excrement matters it receives into a homogenous fluid, only slightly turbid, and holding all the solid matter in suspension in the form of scarcely visible filaments. The vault is self emptying and continuous in its workings."

The same authors reported that septic tanks were introduced in North America in 1884 with the design of a two chamber tank utilizing an automatic siphon for intermittent effluent disposal. Their use provided the convenience and safety of interior plumbing for rural areas but environmental problems associated with their use arose in more densely populated areas. The common misconception, both then and today, that the septic tank is a "self-emptying and continuous process" has led to many septic tank and disposal field failures.

Design criteria for septic tanks have remained basically unchanged during their 100 years of formal history. Their main function is the removal, from waterborne wastes, of solids which might otherwise plug soil voids in a receiving subsurface disposal area. The waste liquid is retained in a reservoir for a specific period of time, thus permitting the heavier sewage solids to settle and form a blanket of sludge at the bottom of the tank. Lighter solids, including fats and greases, rise to the surface and form a scum layer. The retained sludge and scum undergo partial digestion and liquefaction. A residue consisting of undigested scum and settled sludge, accumulates in the tank and must be removed periodically by properly cleaning the tank. The solids and liquids removed in the cleaning process are called septage.

In a properly operating septic tank, a liquid effluent low in suspended solids is discharged to a subsurface soil disposal system. As this wastewater percolates downward through the soil, the physical, chemical, and biological reactions within the soil matrix further remove wastewater constituents before the liquid reaches the water table.

The septic tank drainage field system, though relatively simple in concept, involves many complex and largely uncontrollable physical, chemical and biological processes. Performance of these systems is a function of many variables including the design criteria used, the construction techniques employed, the characteristics of the wastewater being treated, site conditions such as areal geology and topography, the physical and chemical composition of the soil mantle, and the attention given to proper maintenance of the septic tank by cleaning out the accumulated solids.

1.2 MAJOR COMPONENTS AND THEIR FUNCTIONS

The simplest septic tank - drainage field system in use today is illustrated schematically in Figure 1.1. Waterborne waste from a single building is conveyed by gravity to an adjacent single compartment septic tank for treatment prior to gravity disposal to a soil disposal field. A number of variations to this basic system have evolved in an effort to improve performance and reliability and to meet local design requirements and environmental constraints. Each of these variations and their designs will be described in detail in subsequent sections of this report.

1.2.1 Collection Systems

The "collection system" refers to the means by which liquid wastes are transferred from their source to the septic tank - disposal field system. In the simplest system,

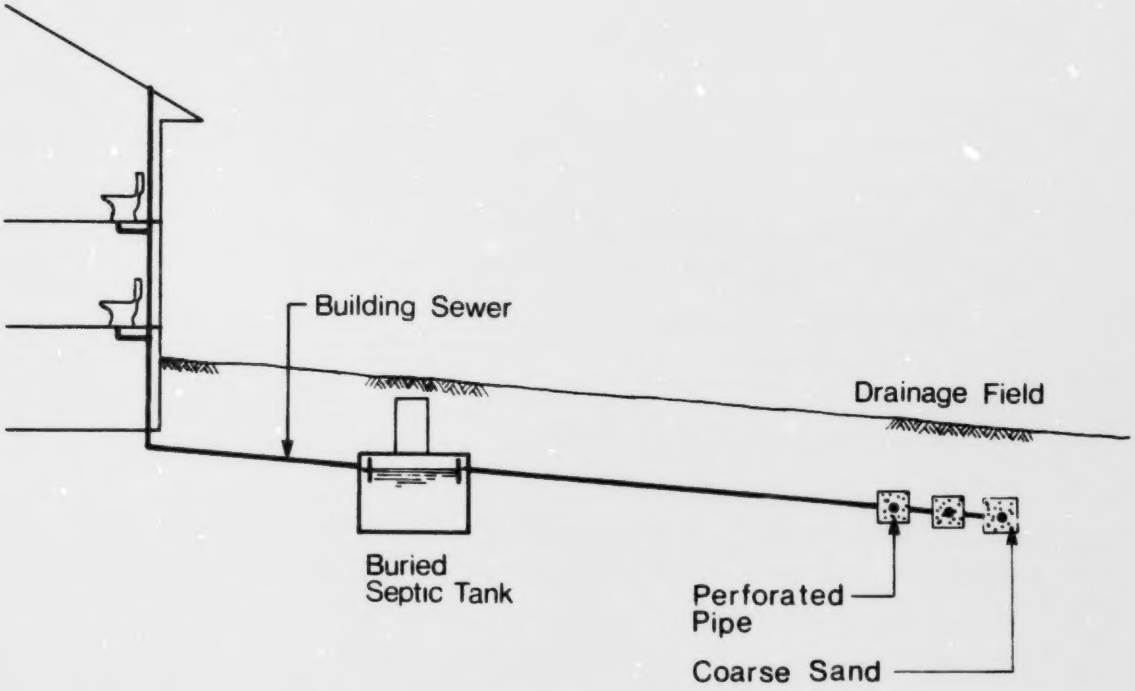
Building Sewer
Drainage Field
Buried Septic Tank
Perforated Pipe
Coarse Sand

Figure 1.1

THE BASIC SEPTIC TANK DRAINAGE FIELD SYSTEM

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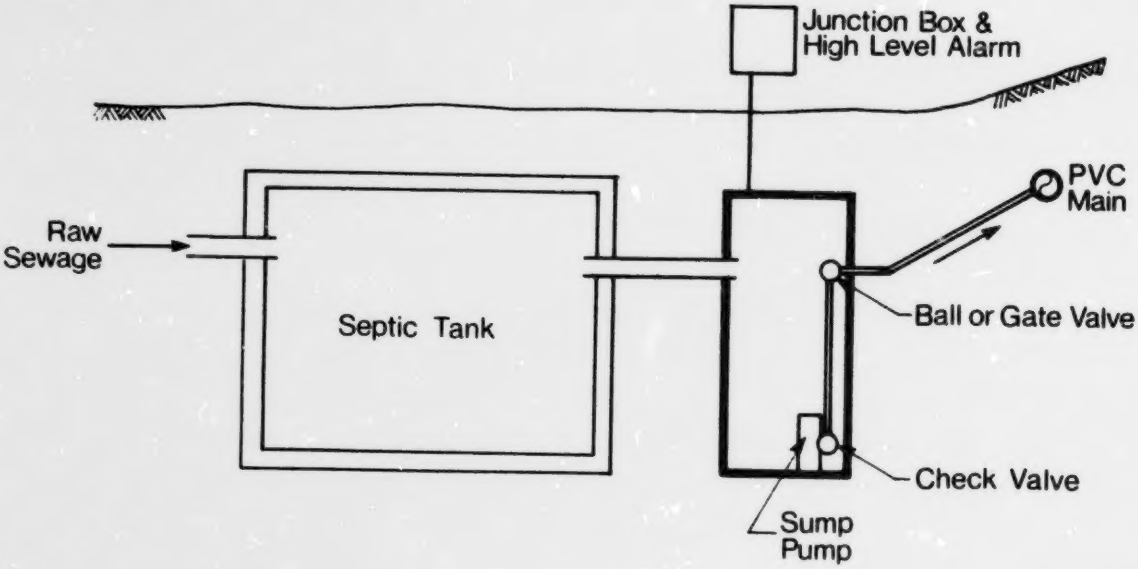
sewage from a single household or multi-unit facility is conveyed by gravity to a buried septic tank. This provides a maintenance free collection system, and burial of the septic tank reduces the risk of freezing in winter. This same gravity collection system is sometimes used where housing units are scattered over a wider area but where a single septic tank - disposal field system is more suitable than individual systems.

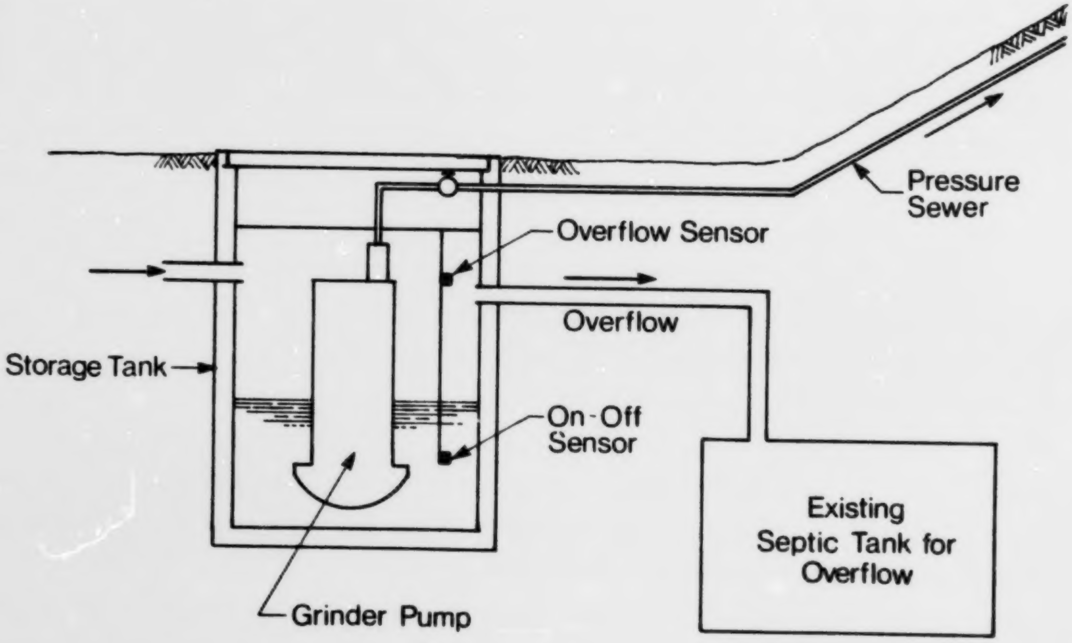
A variation of the conventional gravity sewer which has been growing in popularity is the small diameter gravity sewer reported by Otis (1984). Small diameter (minimum 100 mm) lines collect settled wastewater from septic tanks which are installed upstream of each connection. With the grit, grease and settleable solids removed, the sewer need not be designed to maintain a minimum flow velocity for self-cleansing. With reduced risks of blockages, the conduits can be laid with a curvilinear alignment and a variable or inflective gradient. This reduces construction costs because the natural topography can be followed fairly closely and most obstructions in its path can be avoided without installing costly manholes.

In many cases, the scattering of housing units, high groundwater or difficult terrain make gravity sewer construction prohibitive. Pressure and vacuum sewers are used as alternatives in such cases. Pressure systems pump effluent from holding tanks or preliminary treatment units either to a main pressure line or to an existing gravity line. Small diameter (5 to 15 cm) plastic or PVC pipe carries the sewage under low pressure from a pump. Piping is laid parallel to the ground surface just below the frost line. Benefits of pressure collection systems include relatively low installation cost and less susceptibility to groundwater infiltration. This is offset by the need for mechanical pumping equipment and associated controls.

The two basic types of pressure collection systems are illustrated in Figure 1.2. The first, referred to as a "STEP (septic tank effluent pumping system)", treats the waste in a conventional septic tank and the effluent is then pumped to a central disposal field. The second, known as the "grinder pump system", consists of a sewage grinder pump within a small storage tank. Waste solids are reduced in size through grinding, before being pumped to the main.


Junction Box &
High Level Alarm
PVC
Main
Raw
Sewage
Septic Tank
Ball or Gate Valve
Check Valve
Sump
Pump

THE "STEP SYSTEM"


Pressure
Sewer
Overflow Sensor
Overflow
Storage Tank
On-Off
Sensor
Existing
Septic Tank for
Overflow
Grinder Pump

THE "GRINDER PUMP SYSTEM"

Figure 1.2

PRESSURE COLLECTION SYSTEMS (USEPA, 1980)

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An alternative to pressure collection is the vacuum system, illustrated in Figure 1.3. Sewage is pulled through collector lines to a central location by vacuum. A special valve in the house connection is designed to separate the atmospheric pressure from the vacuum in the main. When sufficient sewage collects behind the valve, it opens and wastewater and air are pushed into the vacuum main. A successive series of plug reformations allows the vacuum in the collector lines to pull the sewage to the collection tank.

The applicability of vacuum systems is fairly site specific. It is efficient on flat or gently rolling terrain and is suitable where homes are dispersed over a wide area or where groundwater is close to the surface.

The design of on-site collection systems is discussed in more detail in Section 2.

1.2.2 Septic Tank Systems

The US EPA (1980) has defined septic tanks as "buried, watertight receptacles designed and constructed to receive wastewater from a home, to separate solids from the liquid, to provide limited digestion of organic matter, to store solids and to allow the clarified liquid to discharge for further treatment and disposal". A single compartment septic tank is illustrated in Figure 1.4. The settleable solids accumulate as sludges on the bottom of the tank while flotable material, consisting primarily of fat and grease, form a scum layer on the top. The partially clarified liquid flows through an outlet structure designed to retain the scum layer inside the tank on the theory that if allowed to escape to the drainage field, fats and greases may clog the soil pores.

The same authors have reported the factors which affect the performance of septic tanks including their size, materials of construction and frequency of maintenance. The tank must be sized to remove suspended solids and flotable materials under all the various hydraulic and organic loading conditions which might be encountered. Septic tanks must be watertight, structurally sound and durable. The accumulated sludges should be pumped out at intervals usually established by routine inspection. Failure to

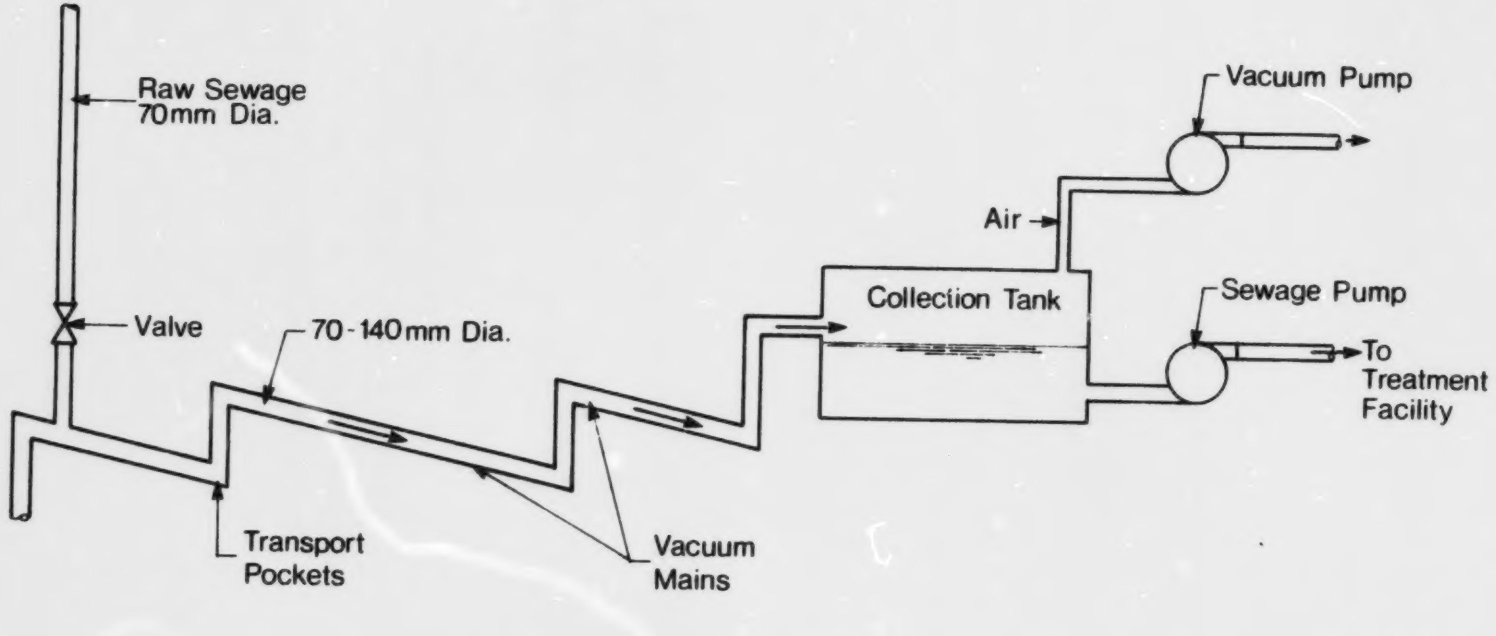

Raw Sewage
70mm Dia.
Valve
70 - 140mm Dia.
Transport
Pockets
Vacuum
Mains
Collection Tank
Air
Vacuum Pump
Sewage Pump
To
Treatment
Facility

Figure 1.3

THE VACUUM COLLECTION SYSTEM

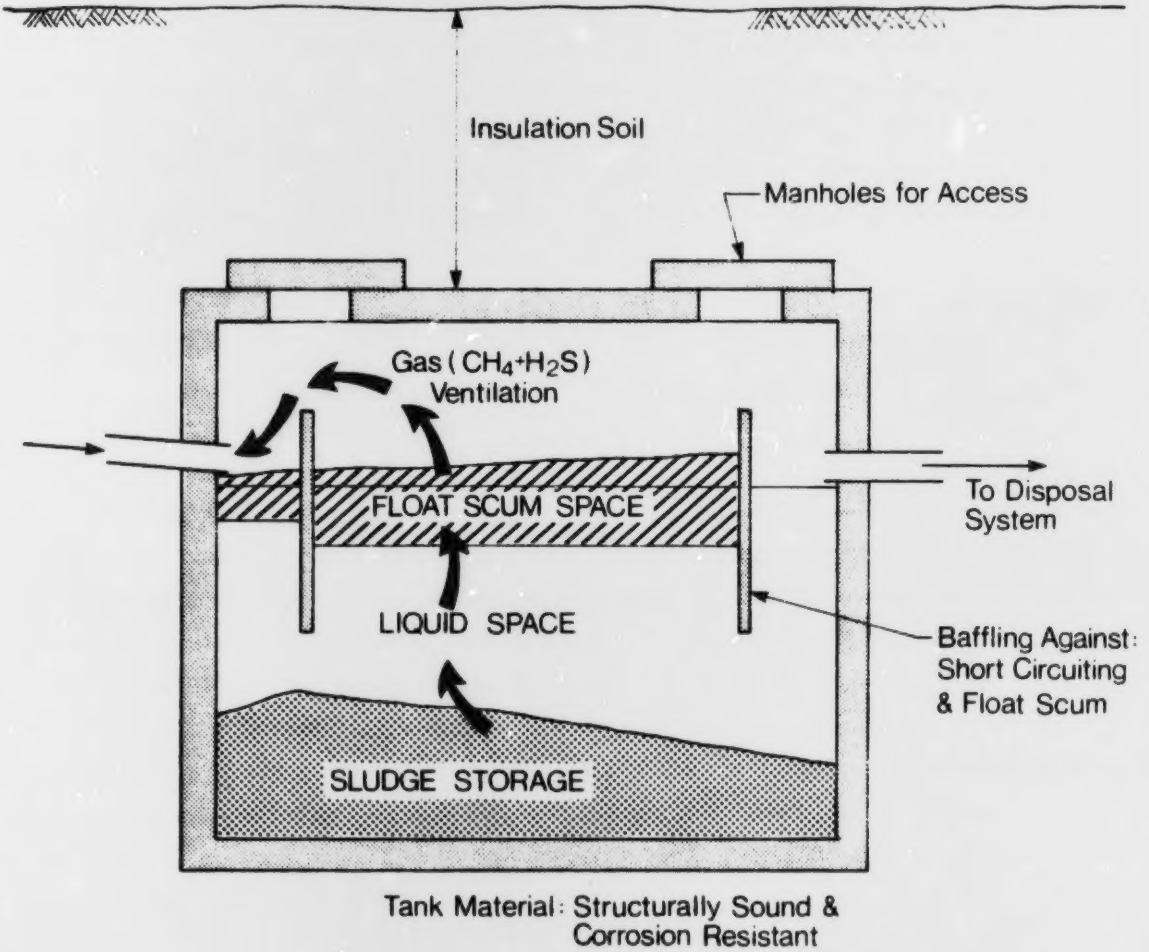

Insulation Soil
Manholes for Access
Gas (CH_4+H_2S) Ventilation
FLOAT SCUM SPACE
To Disposal System
LIQUID SPACE
Baffling Against: Short Circuiting & Float Scum
SLUDGE STORAGE
Tank Material: Structurally Sound & Corrosion Resistant

Figure 1.4

ESSENTIAL COMPONENTS OF A SEPTIC TANK

adhere to prudent management practices has been a frequent cause of disposal field failures.

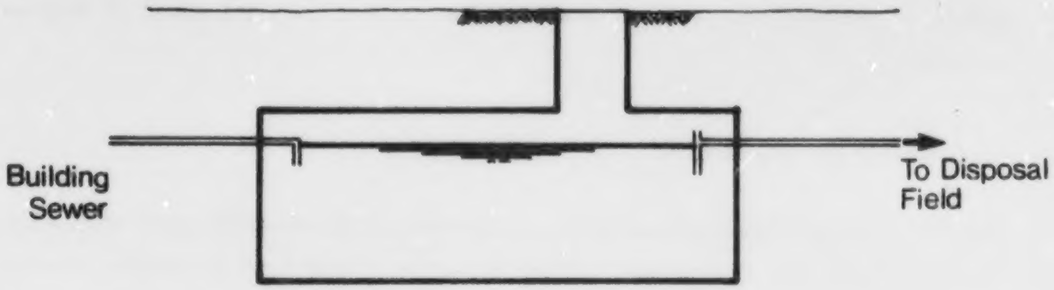
1.2.3 Distribution Systems

Numerous authors have stated that the method of distributing septic tank effluent to the disposal field influences soil bed performance. Their findings will be referred to in Section 4. The simplest distribution system, commonly known as the trickle flow system, is one in which effluent leaves the tank as raw sewage enters the tank. Loading to the disposal field is in response to the rate of sewage generation. A preferrable gravity distribution system is one in which the effluent is loaded to the disposal field in slug doses with a syphon device on the outlet from the septic tank. A third alternative makes use of a low pressure pump to dose the disposal field at intervals comparable to a syphon system. Pump distribution is usually required where depth of burial of the septic tank prohibits the use of a gravity system. Figure 1.5 illustrates the three effluent dosing alternatives.

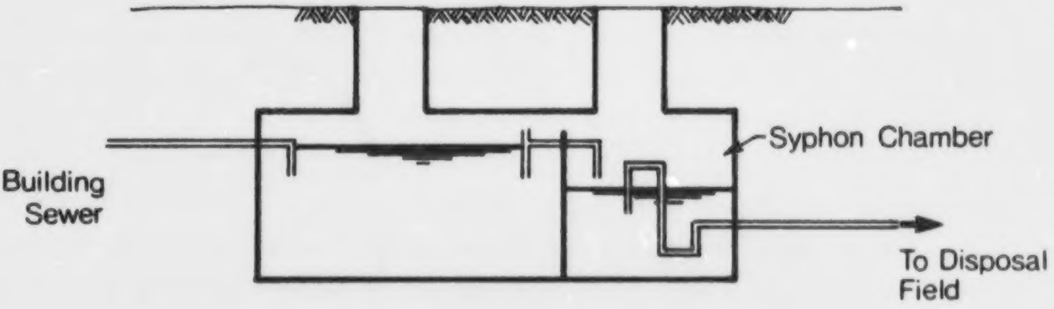
In designing an effluent distribution system the primary obecjtive is the uniform application of liquid over the entire disposal field at a rate lower than the long-term acceptance rate of the soil. Many different dosing systems have been used in disposal field systems, including:

1) 100 mm diameter perforated pipes;
2) pressure distribution networks;
3) nylon fabric wrapped corrugated tubing; and
4) specific proprietary designs (discussed in Section 4.2.4).

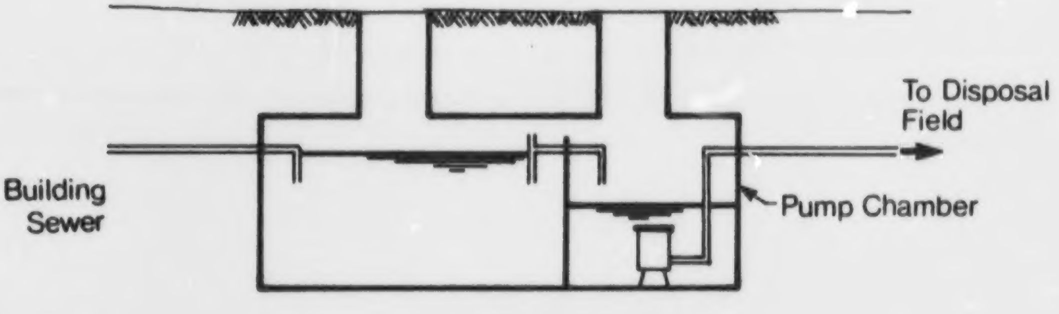
The choice of a particular dosing system depends upon site and operational considerations.


Building Sewer
To Disposal Field

TRICKLE FLOW


Syphon Chamber
Building Sewer
To Disposal Field

SYPHON DISTRIBUTION


To Disposal Field
Building Sewer
Pump Chamber

PUMP DISTRIBUTION

Figure 1.5
SEPTIC TANK EFFLUENT DISTRIBUTION ALTERNATIVES

1.2.4 Soil Disposal Beds and Trenches

Subsurface soil absorption is usually an excellent method for treatment and disposal of wastewater from septic tanks since soil is a superb treatment medium for sewage effluent. Physical chemical and biological processes in the soil provide effective treatment and polishing of septic tank effluent as it percolates to the groundwater.

Soil disposal systems commonly used in North American practice include:

1) conventional trenches;
2) beds; and
3) disposal mounds.

Trenches and beds are shallow, level excavations in the natural soil. The bottom of the excavation is filled with 15 cm or more of clean rock or gravel over which the wastewater distribution piping is laid. Additional rock or gravel is placed over the pipe and covered with a semi-permeable barrier to prevent the backfill from penetrating the rock.

A disposal mound consists of trenches or beds constructed in suitable fill material placed above the natural soil surface. Septic tank effluent is pumped or siphoned into the mound (usually medium to coarse sand) through a pressure distribution network. Mounds are usually used in relatively impermeable soils or in soils with shallow water tables or shallow creviced or porous bedrock.

The criteria for selecting suitable soils for drainage fields, the methods of evaluating on-site disposal field design criteria, the selection of the most suitable absorption systems and the operating characteristics for those systems have all been the subject of numerous studies during the last several decades. Winter climatic conditions particularly affect the design and operating criteria for disposal field systems. These effects are discussed in Section 4.

1.3 CURRENT APPLICATIONS

In Canada today, the majority of septic tank - disposal field systems are used for the treatment and disposal of sewage from single-family housing units. The recent trend towards low energy, cost-effective sewerage systems is resulting in the increased use of these systems for small urban or semi-rural housing developments or public facilities. Septic tank - disposal field systems also have industrial applications particularly for treating biodegradable wastewaters from food processing operations.

1.3.1 Single Housing Units

Scalf and Dunlap (1977) estimated that approximately one-third of the homes in the United States dispose of domestic wastes through individual on-site disposal units. Approximately eighty-five percent of those units are septic tank - soil absorption systems. Canadian statistics are expected to be similar.

Development of the septic tank - disposal field system has been ideal for rural homes since it has made possible the use of conventional plumbing systems. In the past, however, the design and installation of rural septic tank systems was often based soley on ensuring effluent disposal. The increased density of individual home septic tank - disposal field systems has created a need to refine the limitations and capabilities of these systems, particularly as they relate to the soil and groundwater environments.

1.3.2 Communities and Multi-Unit Developments

With the recent emphasis on least-cost sewage collection and treatment alternatives, the use of septic tanks to service small community housing developments and apartment buildings has become more popular. In many cases, the cost of developing and operating community collection systems are prohibitive when compared with on-site handling and disposal alternatives or a combination of communal and on-site systems. At the same time, there has been a greater willingness on the part of regulatory agencies to permit such installations since applied research studies have recently removed much of the suspicion and mystique associated with septic tank -

disposal field systems. In most cases, rational design procedures are now available and the necessary regulatory standards are in place. Several case studies are discussed in Section 4.3.3.

Various collection, treatment and disposal systems have been developed for small communities using septic tank systems. The simplest consist of gravity collection to a central septic tank - disposal field. Other alternatives include gravity collection to individual homeowner septic tanks, with effluent pumped through low pressure sewers to central disposal fields or disposal by pumping to "cluster facilities" (Guttenplan, 1984). In all cases, the need for a rational design of the soil disposal system cannot be overemphasized. Failure to do so may cause hydraulic failure or groundwater contamination and subsequent hazard to human health.

1.3.3 Industrial Applications

The application of septic tank - disposal field systems in disposal of effluents from industrial sites requires careful consideration of wastewater characteristics. Firstly, these systems are only suitable for readily biodegradable wastewaters. Septic tanks have been used successfully in meat packing plants and small fruit and vegetable and dairy processing plants. Effluents from these plants are very concentrated but easily broken down by anaerobic septic tank bacteria and soil aerobic microorganisms. Secondly, the treatment and disposal facilities should be sized in accordance with wastewater strength. Failure to design the system according to wastewater characteristics has often accelerated the failure of soil disposal beds. Effluent characteristics of particular concern include oil and grease concentrations and oxygen demand.

1.4 SUMMARY

Septic tank - disposal field systems are used quite successfully to treat and dispose of wastewater from single family dwellings, small community developments, and certain food processing industries. A number of design options are available for both the septic tank - soil disposal system and the wastewater collection and distribution systems. The selection and design criteria for particular options depend on service requirements and local conditions. In most cases, the selection and design of septic tank - disposal field systems are based on rational standards developed by regulatory agencies. With proper design and operation, these systems can provide safe and long term wastewater treatment and disposal.

SECTION 2

COLLECTION SYSTEMS

2.1 INTRODUCTION

Wastewater collection systems range in complexity from simple gravity collection from a single household or building to community collection using gravity, pressure, or vacuum collection systems or combinations thereof. The design criteria for each type of collection system will vary with specific site conditions and design concepts.

The gravity sewer is by far the oldest, most common, and most reliable system, even when lift stations must be used due to natural low spots in the collection system. Vacuum and pressure collection systems are relatively recent developments and have application in special circumstances when conventional approaches are either too costly or technically impractical. In designing a sewage collection system, the amount of sewage that must be handled and treated over the design life of the system should be quantified and the following information collected:

1) population to be served (25 year projection);
2) per capita sewage flows;
3) type of system input (domestic, commercial, industrial);
4) groundwater levels;
5) local soil conditions;
6) total land area to be served (based on 25 year projection);
7) capacity of the sewage treatment system; and
8) location of water supply system components.

It is important to design the collection system to meet the long-term needs (25 to 50 years), even though the treatment system is usually designed on a shorter-term basis.

2.2 GRAVITY COLLECTION

The major advantage of gravity systems is reliability. Well-designed and constructed gravity sewers usually last 50 years or more and require minimal maintenance. Gravity systems are usually very simple in that no mechanical equipment is needed. The five types of pipe used include plastic, concrete, vitrified clay, asbestos cement, and cast iron.

In order to provide access for maintenance, manholes are installed at intervals of 100 to 150 m. Manholes provide physical access for removing scale or grease accumulations with mechanical cleaning equipment and for unclogging plugged sewers. Manholes also allow air to enter the sewer thereby reducing the likelihood of anaerobic conditions which can result in hydrogen sulphide gas production and associated corrosion problems.

In some collection systems, sewage is gravity collected at a local low point and then pumped using either wet or dry pit sewage lift stations to the central septic tank or disposal field or a main collector at a higher elevation. Design details and standards for gravity collection systems are presented in a design manual entitled "Recommended Standards for Water Supply and Sewerage" published by Alberta Environment, Standards and Approvals Division.

Otis (1984) presented case studies which presented design criteria, construction costs and performance data of small diameter gravity sewer systems. Originally developed in South Australia, they have been reported in use for small communities in the United States since 1977.

2.3 PRESSURE COLLECTION

The first documented attempt to use a pressure collection system was reported by Clift (1968) but the system described did not employ current standard design techniques and materials. A pressure collection system is the reverse of a water distribution system. While the latter employs a single inlet pressurization point and a number of user outlets, the pressure sewer embodies a number of pressurizing inlet points and a single outlet.

The two major elements of any pressure system include the on-site or pressurization facility and the primary conduit or pressurized sewer main. There is a wide divergence of opinion on the proper design of the pressurization facility due to the number of different proprietary mechanical devices available for use. In all cases, however, household wastes are collected in the building drain and conveyed to the pretreatment or pressurization facility.

In a STEP system, household wastewater is collected by the building drain and transported to a septic tank. Septic tank effluent then flows to a receiving tank which houses the pressurization device, control sensors, and required valving. Design decisions related to effluent holding tank installations include materials of construction, dimensions and operating levels for the tank, pump choice, and ancillary needs. Since the effluent holding tank follows the septic tank, the tank inlet must be below the septic tank liquid level to allow gravity flow and in most cases the holding tank is located as close as possible to or made an integral part of the septic tank.

The two most important design concerns with pressure collection systems are the possibility of power failures and the ability of the collection system to handle pump malfunctions. Environment One Corporation (1973) conducted an inventory of power outages in the United States for the years 1968 to 1972. They showed that outages greater than nine hours in duration were caused by major natural disasters such as floods, hurricanes, and earthquakes. They recommended the use of a nine hour period for design purposes. It is reasonable to assume that no more than 50 percent of the total daily household flow would occur in that time period (US EPA, 1977). This volume could be handled by the freeboard capacity typically available in septic tanks. In rural areas, the loss of power in households served by individual water supplies also eliminates the possibility for wastewater generation since water supplies also become inaccessible. In addition to power outages, system design should also account for downtime to make equipment repairs. For community collection systems, a number of replacement pumps should be available for quick repairs to the system.

A grinder pump system consists of a sewage grinder pump within a small holding tank. Coarse solids in the raw sewage are reduced in size through grinding as the sewage is pumped to the collector main. Grinder pump installations are used in community collection systems where a central septic tank is utilized. Advantages over the STEP system include:

1) reduction in the number of septic tanks and the associated on-lot piping;
2) the flexibility to install pumping units in the basement for ease of maintenance; and
3) less severe operating conditions.

Since grinder pumps tend to macerate many of the coarse solids which would otherwise settle rapidly in the septic tank due to their size and density, it is reasonable to suggest that septic tank design should accommodate decreased solids settling rates.

Grinder pump holding tanks are too small to provide emergency storage of sewage, and so to accommodate power outages or pump failures, the following may be employed:

1) absorption pits and beds that percolate grinder pump holding tank overflow into the ground;
2) an emergency storage tank located between the house and the grinder pump unit;
3) standby power; and
4) an interconnection with an adjacent grinder pump unit.

2.4 VACUUM COLLECTION

Vacuum systems depend on a central vacuum source constantly maintaining 350 to 750 mm of mercury on small-diameter collection mains. A gravity vacuum interface valve separates atmospheric pressure from the vacuum in the mains. The valve can be located either in the home sanitary sewer service line or in the various household plumbing fixtures. The interface valve opens for a preset interval at which time a volume of sewage enters the main, followed by a volume of atmospheric air. The packet of liquid, called a slug, is propelled into the main by the differential pressure of vacuum in the main and the higher atmospheric pressure behind the slug. After a distance, the slug breaks down by shear and gravitational forces, allowing the higher pressure air behind the slug to slip past the liquid. With no differential pressure across it, the liquid then flows to the lowest elevation, and vacuum is restored to the interface valve for the subsequent operation. When the next upstream interface valve operates, identical actions occur, with that slug breaking down and air rushing across the second slug. That air then impacts the first slug and forces it further down the system. After a number of operations, the first slug arrives at the central vacuum source. When sufficient liquid volume accumulates in the collection tank at the central vacuum source, a transfer device, such as a sewage pump, delivers the accumulated sewage to the sewage treatment facility. Figure 2.1 illustrates the typical layout of a vacuum collection system. The contents of the vacuum collection system are usually transferred to the septic tank with the use of non-clog sewage pumps having sufficient net positive suction head (NPSH) to overcome tank vacuum.

The following summary on the application of vacuum systems was given by Johnson (1978):

1) The energy requirements to maintain a constant vacuum are relatively high.
2) The pumps are relatively expensive.
3) The system is easy to maintain by a home owner since the valve is the only moving part.

2.5 COLLECTION SYSTEM SUMMARY

The choice of collection system can influence both the design and performance of septic tanks and soil disposal fields. Table 2.1 provides a summary of the nature of these influences for each of the aforementioned collection systems.

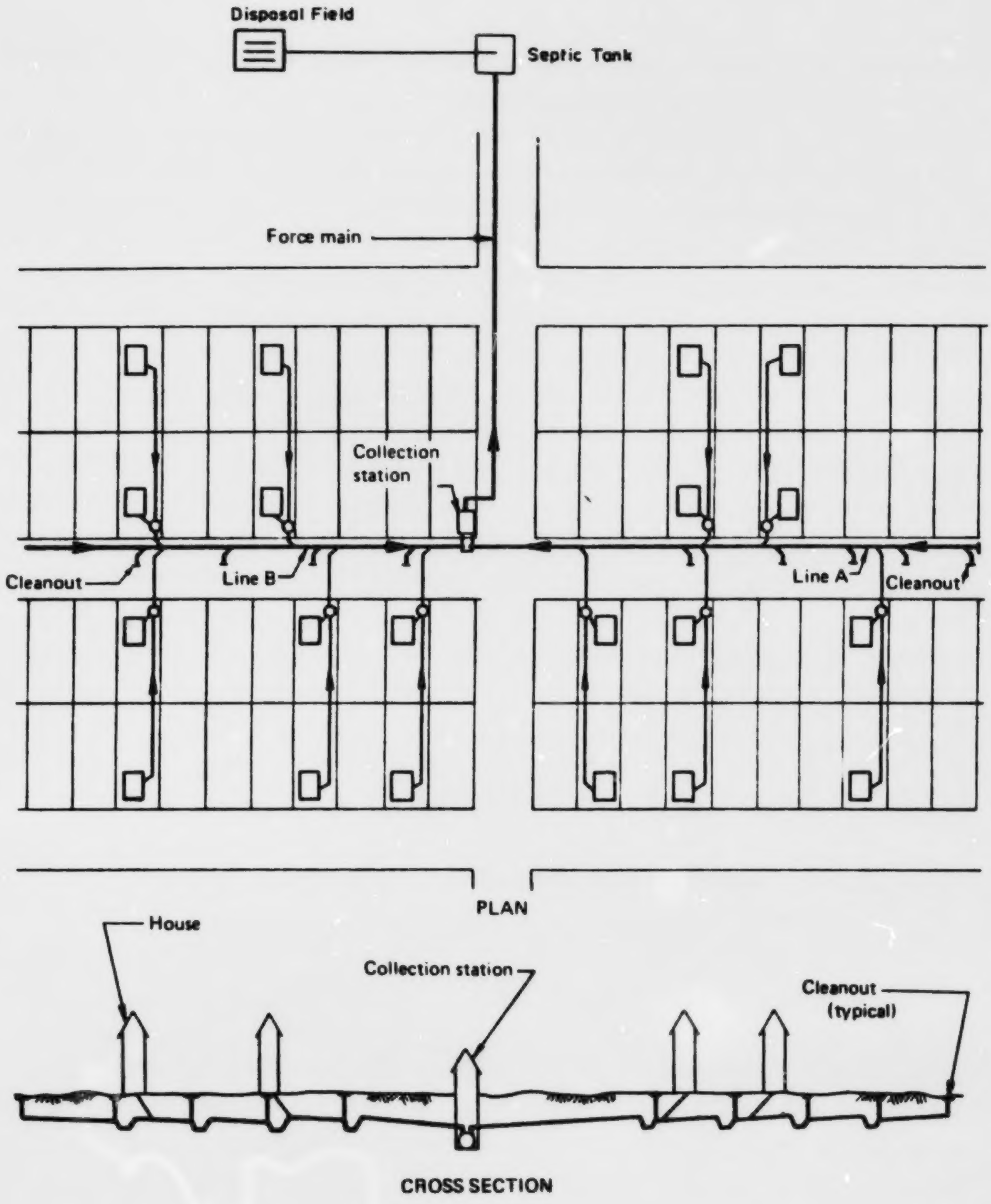

Disposal Field
Septic Tank
Force main
Collection station
Cleanout
Line B
Line A
Cleanout
PLAN
House
Collection station
Cleanout (typical)
CROSS SECTION

Figure 2.1

TYPICAL LAYOUT
VACUUM SEWER SYSTEM (USEPA, 1980)

TABLE 2.1

COLLECTION SYSTEM INFLUENCES ON SEPTIC TANK - DISPOSAL FIELD DESIGN AND PERFORMANCE

Type of Collection System	Factors of Significance
Gravity Collection	1) Storm run-off water and sewer infiltration water may form a significant component of the total flow.
	2) The physical characteristics of suspended solids entering the septic tank are influenced by the presence of mechanical lift stations.
	3) The flow regime entering the septic tank is influenced by the presence of mechanical lift stations and the loading may be very intermittent depending on lift station design.
	4) Where no lift stations are used, the depth of burial of the septic tank is controlled by the required depth of burial of the sewer.
Pressure Collection	1) Suspended solids are usually macerated by mechanical pumps thereby creating a homogenous mixture.
	2) The number of septic tank - disposal field units required may be a function of the type of pressure collection system used (STEP or grinder pump system).
	3) Loading to the septic tank is usually in slugs in accordance with pumping frequencies.
	4) Back-up systems are required in the event of power failure.
Vacuum Collection	1) Suspended solids may be macerated by mechanical pumps.
	2) Vacuum collection systems usually included low water-use fixtures. Sewage volumes are thereby reduced and sewage strength is relatively high.
	3) Loading to the septic tank is usually in slug doses.

SECTION 3

SEPTIC TANK DESIGN PRACTICES

3.1 SITE SUITABILITY EVALUATIONS

In most cases, the effluent from a septic tank is discharged to a soil disposal system, and the suitability evaluation of the site for such a system is generally the first step in the overall design. As there are few constraints to the use of the septic tank itself, it is generally soil disposal system considerations that determine the overall suitability of a site for a septic tank - disposal field system. Those factors which must be considered when evaluating the appropriateness of using such a system include:

1) characteristics and amount of the wastewater to be treated;
2) size and slope of the building lot(s);
3) location of the systems in relation to streams, wells and structures;
4) character and depth of the soil mantle;
5) depth to saturated soil conditions, fractured bedrock, etc; and
6) climatic conditions.

Further details regarding site suitability evaluations are discussed in Section 4.

3.2 MAJOR SEPTIC TANK COMPONENTS AND THEIR DESIGN

Septic tanks are normally the first stage or process of an on-site treatment system. The main function of a septic tank is to produce an effluent that can be discharged to a soil disposal system. A septic tank should therefore be designed to remove close to 100 percent of the settleable and flotable solids and to provide partial anaerobic decomposition of the organic solids. Tank design should provide the following (Baumann et al., 1977):

1) adequate sludge storage so as to prevent settleable solids from flowing into the outlet pipe;

2) scum storage;

3) inlet baffle to direct the flow downward and to prevent the scum layer from plugging the house sewer;

4) outlet baffle to retain the floating scum layer and prevent scum from entering the outlet pipe;

5) venting to allow the escape of methane and hydrogen sulfide; and

6) access to the tank interior for inspection and cleaning.

These same authors suggested that a properly designed septic tank system should be able to achieve the following treatment efficiencies when treating typical household wastes:

1) 98 to 99 percent removal of settleable solids;
2) 65 to 80 percent removal of the five-day biochemical oxygen demand (BOD_5); and
3) 60 to 90 percent removal of suspended solids.

Design factors which affect the performance of a septic tank include:

1) tank geometry;
2) hydraulic loading characteristics;
3) inlet and outlet arrangements;
4) number of compartments; and
5) operation and maintenance practices.

The US EPA (1980) reported that septic tanks for single-family homes are usually prefabricated, ready for installation and are normally designed in accordance with local codes. For larger residential communities, institutions or commercial applications, local codes often define a series of rational design criteria for selection and sizing of septic tanks.

3.2.1 Tank Volume

Weibel et al. (1954) concluded that a septic tank should have a "liquid volume sufficient for a 24-hour fluid retention time at maximum sludge depth and scum accumulation". Local regulatory agency codes for tank sizing are usually based on estimated daily flows. Salvato (1972) summarized these recommendations (see Table 3.1). The table illustrates that hydraulic design codes for smaller septic tanks have been based on the number of bedrooms per home and the average number of persons per bedroom. (MERL, 1978) indicated that the average per capita wastewater contribution particularly in rural areas was 170 L/d. To estimate daily sewage flow, a value of 285 L/d per person is coupled with an assumed dwelling density of two persons per bedroom. The proposed septic tank volume therefore provides a safety factor of two to three times the daily design flow. As solids accumulate, however, the detention time decreases. Solids must be removed before the detention time is reduced to less than 24 hours.

TABLE 3.1

TYPICAL RECOMMENDED SEPTIC TANK SIZES

(Salvato, 1972)

Population		Sewage	Septic Tank-Minimum (Incl. Garbage Grinder)			
Bedrooms	Persons	Flow (L/d)	Length (mm)	Width (mm)	Depth (mm)	Volume (L)
2	4	1140	2300	1100	1200	2840
3	6	1520	2600	1100	1200	3400
4	8	2270	2700	1200	1200	3790
5	10	2840	3400	1200	1200	4730

Jones (1975) discovered from a water use study of 22 homes (10 on a public water supply and 12 on private water systems) that mean daily water use was 185 L per person with a range between 120 and 250 L per person. Most of the hydraulic loading to a septic tank was found to occur in a short period of time - 25 percent in one hour or less and 45 percent in four hours or less. He further speculated that "in today's homes, which have many automatic water-using appliances, the peak demand is often created by the simultaneous use of the automatic washer, dishwasher and kitchen sink."

Baumann et al., (1977) explained why the rate at which water is used in the home is not the rate at which water enters and leaves a septic tank. First, the house plumbing and the sewer connection pipe to the septic tank provide flow damping since they must be filled to a certain level before flow begins. Therefore, the time of discharge is lengthened and the peak discharge rate is reduced. Secondly, the septic tank itself provides flow damping. Before discharge from the tank can begin, the liquid level in the tank must rise above the invert elevation of the outlet pipe. For a given rate of flow into the septic tank the rate of rise in liquid level is inversely proportional to the tank's surface area. This rise, therefore, serves to damp or attenuate the discharge rate from the tank.

Jones (1975) illustrated these flow attenuation factors graphically with Figure 3.1. The phenomena may be explained as follows:

1) Curve 1 represents the rate of water use in a home with flow ceasing at time t_1.

2) Curve 2 represents the rate at which water is discharged into the septic tank. This rate is lower because of the first reason mentioned above. The water volume represented by Area A is in temporary storage until the rate of outflow from the home (Curve 1) equals the rate of inflow to the septic tanks (the intersection of Curves 1 and 2). At this point the flow into the septic tank reaches a peak value. The volume of storage represented by Area A is now discharged, and is represented by Areas D and E. The peak flow has thus been reduced from Point 1 to Point 2 and the discharge time has been increased to time t_2.

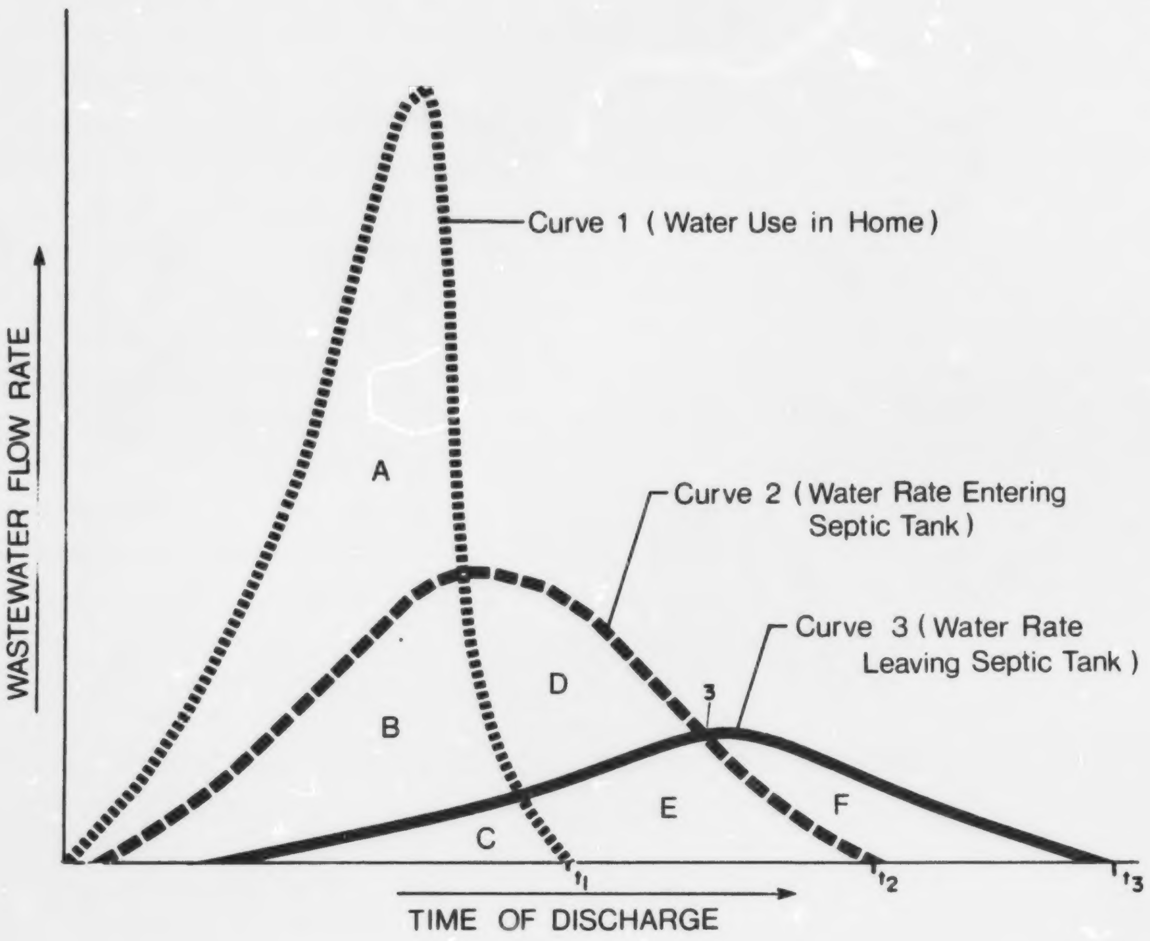


Figure 3.1

FLOW ATTENUATION IN A HOME SEWER & SEPTIC TANK (JONES, 1975)

3) Curve 3 represents the rate at which sewage leaves the septic tank once the liquid level is above the invert of the tank outlet. The discharge rate slowly increases as the water volume represented by Areas B and D is put into storage. At the instant the area represented by B and D is stored, the tank inflow rate is equal to the outflow rate at Point 3 and the discharge rate from the septic tank will begin to decrease. The water volume represented by Areas B and D will be discharged as Area F, and the time of discharge will be extended to time t_3.

This phenomenon illustrates several hydraulic design principles. First, the length of time over which discharge continues and the volume available for storage are functions of both the surface area of the tank and the diameter and slope of the outlet pipe. For each unit rise in sewage level in the tank, the storage volume available is a direct function of surface area. For a given storage depth, the discharge rate is directly proportional to outlet pipe diameter and slope.

Secondly, this hydraulic principle explains the theoretical advantages of dual or multi-chamber septic tanks. The rate of flow from the first compartment of the septic tank into the second would be represented by Curve 3. A lower peak flow rate reduces the hydraulic shock loading, thereby minimizing hydraulic disturbance of the settled suspended solids. Attenuated flow rates also provide a more stable environment for the biological removal of dissolved organics in the sewage. The addition of a third or even fourth compartment provides further attenuation by the same principles as explained above.

In addition to the hydraulic flow considerations, the septic tank should also provide for sludge and scum accumulation. Jones (1975) and Weibel et al. (1954) proposed that the design depth for both submerged scum and sludge storage should be approximately related to tank surface area as summarized in Table 3.2 (Figure 3.2 illustrates the terms used in Table 3.2). Weibel et al. (1954) suggested that in order to keep the scum from entering the outlet pipe (Figure 3.2), a scum clear space of about 10 cm should be provided in all septic tanks when the water level is at the invert of the outlet. The baffle or outlet tee extension should extend high enough to keep scum out of the outlet.

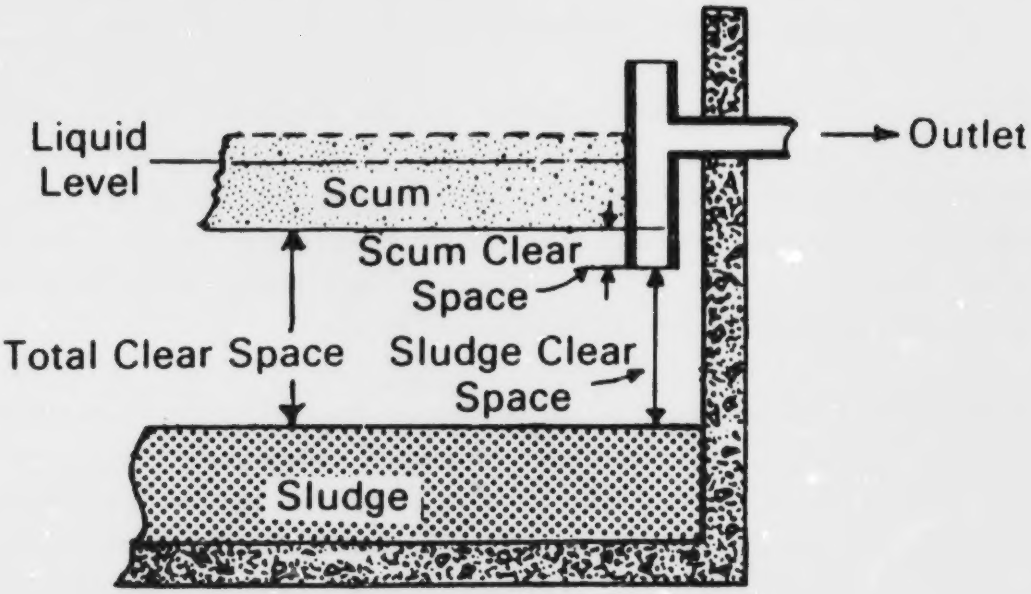

Liquid Level
Scum
Outlet
Scum Clear Space
Total Clear Space
Sludge Clear Space
Sludge

Figure 3.2

SEPTIC TANK SCUM
AND SLUDGE CLEAR SPACES

TABLE 3.2
SCUM THICKNESS AND SLUDGE CLEAR SPACE DESIGN RECOMMENDATIONS

Tank Surface Area (m^2)	Scum Thickness (Jones, 1975) (mm)	Sludge Clear Space (Weibel et al., 1954) (mm)
2.3	250	250
2.8	200	180
3.3	180	150
3.7	150	130
4.2	140	100

Jones (1975) suggested that there was a more preferable basis for establishing design sludge storage depths and these are summarized in Table 3.3. For a given tank surface area, increasing the septic tank volume provides increased efficiency of suspended solids removal by increasing the volume of clear space provided (Figure 3.2).

TABLE 3.3
RECOMMENDED MINIMUM SLUDGE CLEAR SPACE (mm)

	Tank	Surface Area (m^2)					
Bedrooms	Volume (L)	2.2	3.0	3.7	4.5	5.2	6.0
3	3530	650	500	400	300	300	230
4	3790	850	650	500	430	350	300
5	4730	1050	800	650	550	460	400

Large septic tank systems must be able to handle the highly variable flows characteristic of single-family home units, as well as continuous flows of 8 to 16 hours per day. The US EPA Design Manual for On-Site Wastewater Treatment and Disposal Systems (1980) suggested that where septic tanks are being installed with existing facilities, the sewage flow should be metered to obtain average daily flows and flow peaks. If this cannot be done, such as in the case of new facilities, the authors recommended the following design procedures:

1) For housing clusters, the total flow can be estimated based on the expected population and a generation rate of 170 L/d/person.

2) For commercial and institutional applications, the authors provided a series of sewage flow tables to be used to estimate wastewater quantities.

The same manual recommended the following additional design criteria:

1) For flows between 2840 to 5680 L/d, the capacity of the tank should normally be equal to 1½ days wastewater flow.

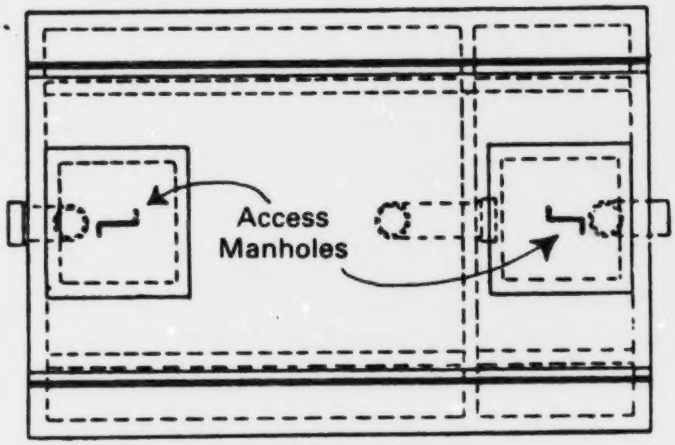
2) For flows between 5860 and 56800 L/d, the minimum effective tank capacity should be established as 4260 L plus 75% of the daily flow.

No rationale were provided for these criteria.

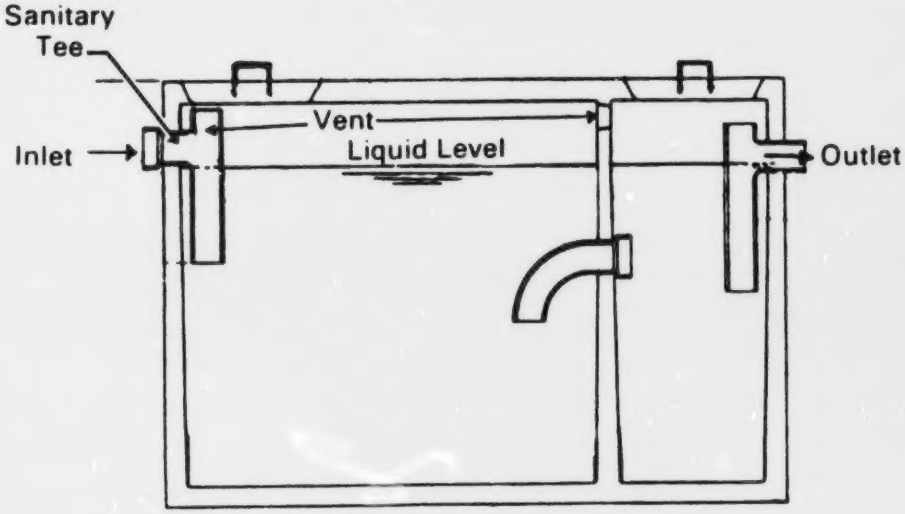
The U.S. Public Health Service (1974) suggested that where garbage grinders are used, additional volume or extra sludge storage space may be desirable to minimize sludge clean-out frequency.

3.2.2 Septic Tank Compartmentalization

Recent trends in septic tank design practice favour the use of double or even multiple rather than single compartmented tanks. A typical two compartment septic tank is illustrated in Figure 3.3. One of the major hydraulic benefits of two compartment septic tanks is the reduction in the peak loadings of wastewater in the final tank,


Access
Manholes

Plan


Sanitary
Tee
Vent
Inlet
Liquid Level
Outlet

Longitudinal Section

Figure 3.3

TYPICAL
TWO-COMPARTMENT SEPTIC TANK

thereby improving effluent quality (see Section 3.2.1). In a dual compartment tank, most of the settleable solids are removed in the first compartment. The second compartment receives the clarified effluent from the first. In order to reduce or prevent intercompartmental mixing due to oscillatory effects, the first and second compartments are usually unequal in size (the second compartment is commonly 1/3 to 1/2 the size of the first), (US EPA, 1980).

Laak (1980) reported the results of various multi-chamber evaluation studies. He evaluated research results from the United States and Australia and reviewed regulations or recommendations from Austria, West Germany, Switzerland and Norway. The author concluded that multi-chamber tanks produce effluents with 50 percent less suspended solids and BOD when compared to effluent from single chamber tanks. In no study did the single chamber tank outperform the multi-chamber tank unless subjected to unequal conditions. The following additional conclusions regarding the design requirements for an efficient septic tank were made by Laak (1980):

1) it should have a detention period longer than 24 hours;
2) the outlet configuration should include a gas baffle;
3) the surface area:depth ratio should be maximized for all chambers to a ratio greater than 2.0; and
4) the interconnections between chambers should be similar in design to the final outlet.

Mara (1981) reported on a prototype design of a three-chamber septic tank which separated toilet wastewater from the remaining household wastewater or sullage. Toilet wastes discharge into the first chamber with overflow to the second chamber while the remaining wastewater discharges directly to the last chamber. The concept is illustrated in Figure 3.4. This separation was recommended in order to avoid excessive dilution of toilet wastes with sullage. The author claimed that the resulting improvements in effluent quality would allow the safe use of septic tank disposal fields on housing areas with densities up to 200 persons/ha. No operating data were provided to support those claims.

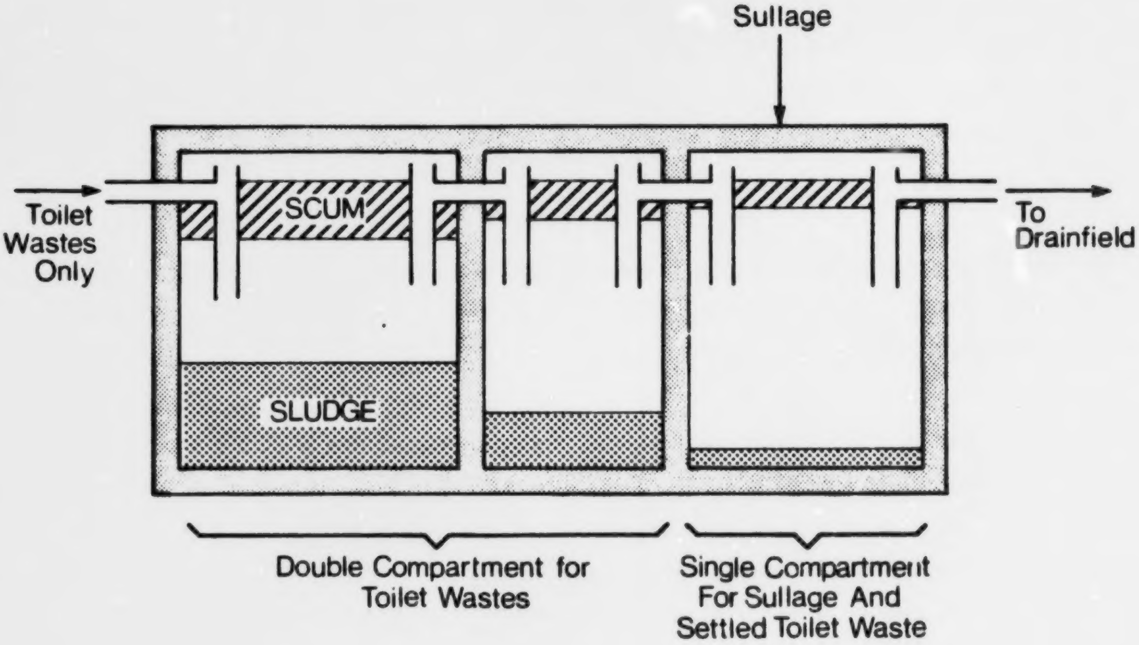

Sullage
Toilet Wastes Only
SCUM
SLUDGE
To Drainfield
Double Compartment for Toilet Wastes
Single Compartment For Sullage And Settled Toilet Waste

Figure 3.4

THREE-CHAMBER SEPTIC TANK WITH SEPARATION OF TOILET WASTES & SULLAGE (Mara 1981)

US EPA (1980) strongly recommended the use of multi-compartment septic tanks for community and commercial applications and indicated that such tanks should have the same design features as single-family home tanks including:

1) compartments separated by walls with ports or slits at proper elevations;
2) proper venting;
3) access to all compartments; and
4) proper inlet and outlet design and submergence.

A multi-chamber configuration similar to that illustrated in Figure 3.5 was also recommended. It consists of a unit of four precast tank sections having single end walls and two compartments.

3.2.3 Inlet and Outlet Structures

The inlet and outlet structures should be designed to minimize the carryover of settleable solids. The US EPA (1980) reported that turbulence and resuspension of suspended solids occur if:

1) the inlet turbulence causes the mixing of the sludge with the wastewater in the clear space;
2) the rise velocity of the water in the vertical leg of the outlet tee resuspends previously settled solids; and
3) the rising gases produced by anaerobic digestion interefere with particle settling and resuspend previously settled solids which are then lost in the effluent.

Baumann et al. (1977) stated that the inlet to a septic tank should be designed to dissipate the energy of the incoming water, to minimize turbulence and to prevent short-circuiting. It should consist of a sanitary tee, an elbow, or a baffle. The invert radius in an elbow or tee helps dissipate energy in the transition from horizontal to vertical flow and prevents dripping that, at the proper frequency, can amplify water surface oscillations and increase intercompartmental mixing. The vertical leg of the

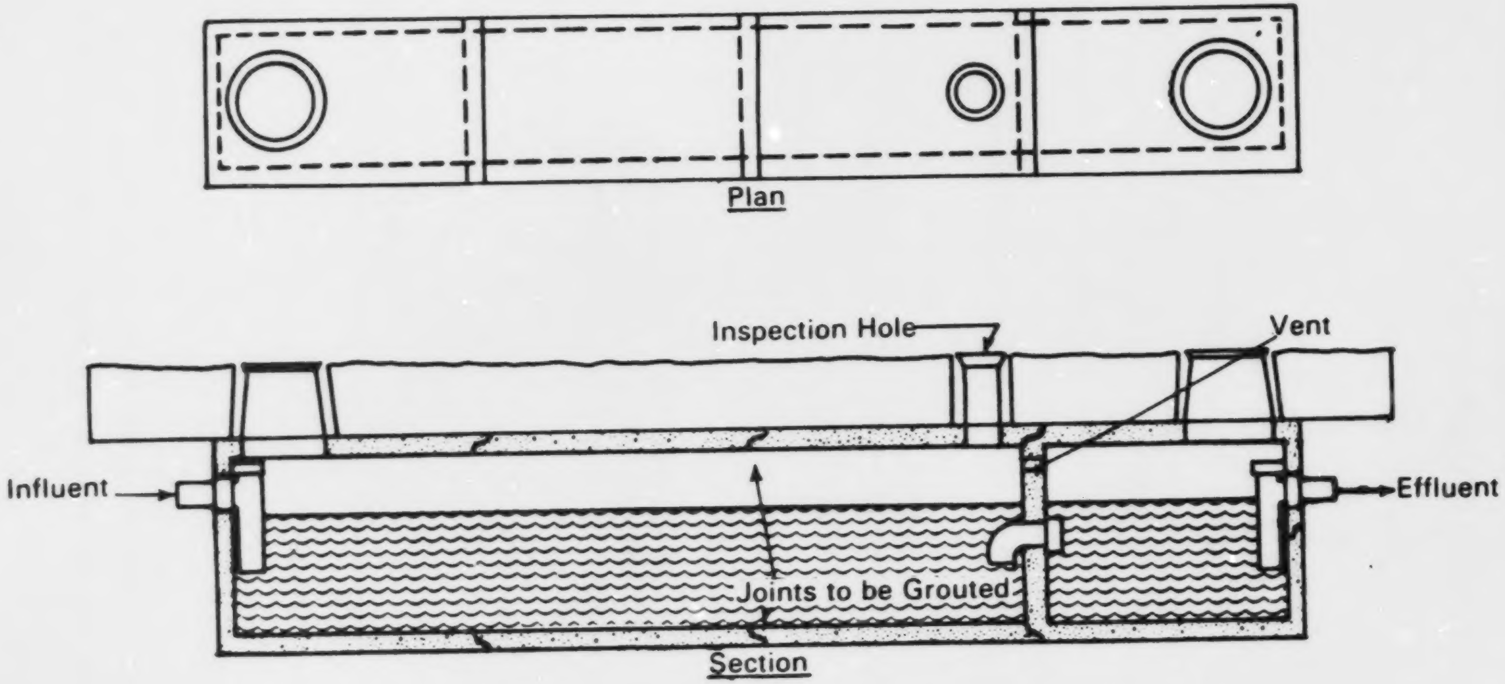

Plan
Inspection Hole
Vent
Influent
Effluent
Joints to be Grouted
Section

Figure 3.5

FOUR PRECAST REINFORCED
CONCRETE SEPTIC TANKS COMBINED
INTO ONE UNIT FOR LARGE FLOW APPLICATION (USEPA, 1980)

elbow or tee should extend below the liquid level in the tank. This minimizes induced turbulence by dissipating as much energy in the inlet as possible.

The US EPA (1980) stated that the outlet structure's ability to retain sludge and scum in either the first or second compartment is a major factor in overall tank performance. Figure 3.6 illustrates several typical outlet configurations using baffles and tees. The authors further stated that the outlet should have both the proper submergence of the bottom pipe and the proper height of the top pipe above liquid level such that the sludge and scum clear spaces balance. Proper venting of sludge gases should also be provided. In an earlier publication, Weibel et al. (1954) recommended outlet tee locations as summarized in Table 3.4. Various types of gas deflection baffles and wedges have been developed to prevent gas-disturbed sludge from entering the rising leg of the outlet. These are illustrated in Figure 3.6.

TABLE 3.4
RECOMMENDED LOCATIONS OF TOP AND BOTTOM OUTLET TEE OR BAFFLE (Weibel et al., 1954)[a]

Total Liquid Tank Capacity (L)	Projection Above Liquid Level	Penetration Below Liquid Level
1890	12%	22%
2840	12%	24%
3790	12%	26%

[a] Refer to Figure 3.3 as reference.

3.2.4 Access and Inspection Ports

Manholes should be provided to allow inspection of the inside of each compartment. They are ususally placed over both the inlet and the outlet to permit cleaning behind

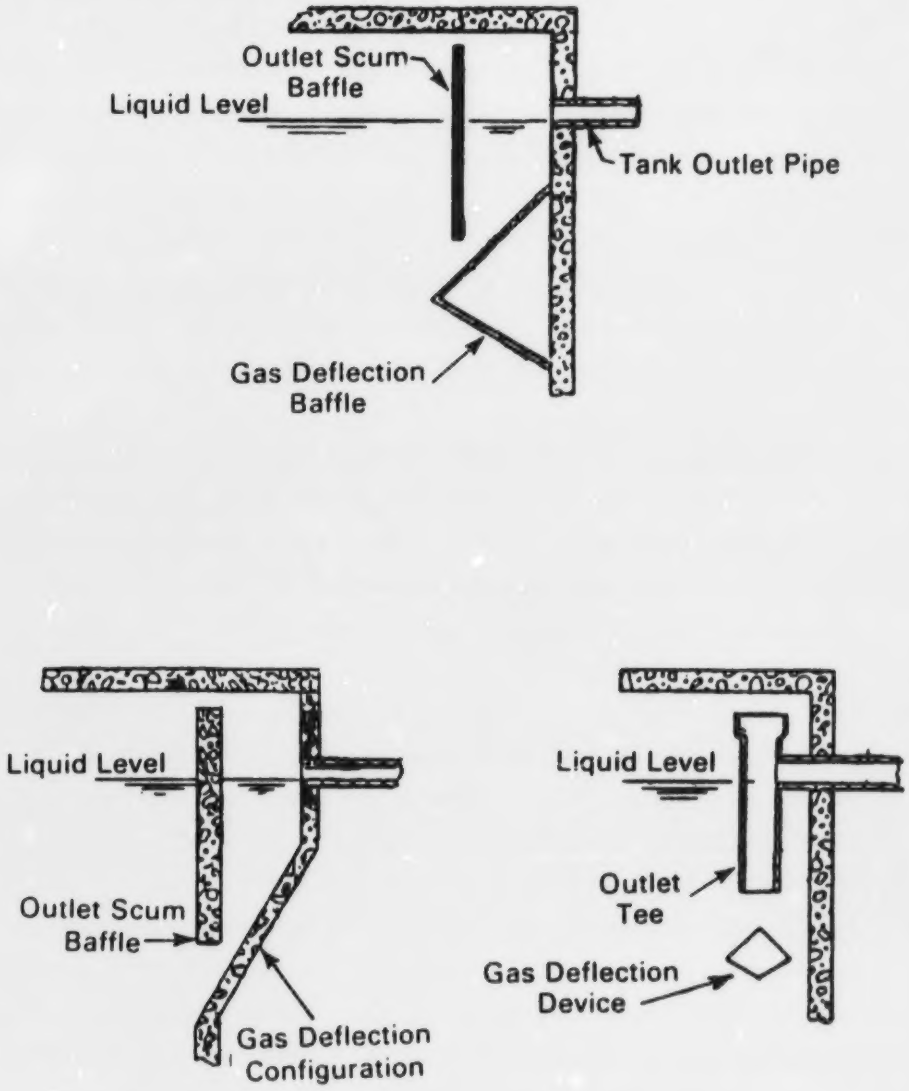

Outlet Scum Baffle
Liquid Level
Tank Outlet Pipe
Gas Deflection Baffle
Liquid Level
Outlet Scum Baffle
Gas Deflection Configuration
Liquid Level
Outlet Tee
Gas Deflection Device

Figure 3.6

TYPICAL SEPTIC TANK OUTLET STRUCTURES TO MINIMIZE SUSPENDED SOLIDS IN DISCHARGE (USEPA, 1980) the baffles. The manhole cover should extend above the actual septic tank to a height not more than 15 cm below finished grade (US EPA, 1980). If the manhole cover extends to the ground surface, a proper seal should be provided to prevent both the escape of odours and accidental entry into the tank. Inspection ports are usually of sufficient size to allow a person to crawl into the septic tank.

3.2.5 Materials of Construction

In a review of present state-of-the-art of septic tank construction practices, the authors of US EPA (1980) made the following observations and recommendations:

1) The most common construction material is concrete. Precast tanks with wall thickness of 8 to 10 cm are usually sealed for watertightness with a few layers of bituminous coating after installation. Care should be taken to seal around inlet and discharge pipes with a compound that bonds both to the concrete and the pipe materials.

2) Steel, treated to resist corrosion, has also been used to construct septic tanks. Despite the use of epoxies or bituminous coatings, steel tanks usually deteriorate at the liquid level and past history has shown that steel tanks have a short operational life, usually less than 10 years.

3) Plastic and fiberglass tanks are very light, easily transported and very resistant to corrosion. Care is necessary during installation and backfilling to prevent structural damage, particularly during cold weather. Because of their light weight, flotation collars are often required to prevent tank flotation in areas with high groundwater potential.

4) Baffles, tees and elbows should be made of durable and corrosion-proof materials. Fiberglass or acid-resistant concrete baffle materials are most suitable. Vitrified clay tile, plastic and cast iron are best for tees and ells. Cast iron inlet and outlet structures should be used in disturbed soil areas where tank settling may occur.

3.2.6 Grease Traps

In certain cases, particularly commercial and industrial applications, the rapid accumulation of the scum layer due to high grease concentrations may be a problem. This grease may be sufficiently bothersome to clog sewer lines and inlet and outlet structures, resulting in restricted flows and poor septic tank performance. Grease traps, installed ahead of the septic tank, remove grease before it can enter the tank.

The US EPA (1980) described the operation of grease traps and summarized current design practices. They reported the following:

1) Grease traps act as small flotation chambers where grease floats to the liquid surface and is retained while the liquid underneath is discharged. Rarely needed in home applications, their main application is in institutions with large volumes of kitchen wastewater such as restaurants, hospitals, cafeterias and industrial camps.

2) Sizing of grease traps is based on wastewater flow and can be calculated from the number and types of contributing sinks and fixtures. A trap should also be rated on its grease retention capacity -- the quantity of grease it can hold before its efficiency drops below 90 percent. Current practice is that grease retention capacity in pounds be at least equal to twice the flow capacity in gallons per minute (US EPA, 1980).

3) Factors affecting the performance of grease traps include wastewater temperature, solids concentrations, inlet conditions, retention time and maintenance practices. By placing the grease trap as close as possible to the source of wastewater, heat loss is reduced and grease separation is facilitated. High solids concentrations can impair grease flotation and cause a solids buildup on the bottom. The use of inlets and outlets similar to those used in septic tanks is recommended to prevent hydraulic surging and grease washout.

The US Public Health Services (1974) suggested some minimum flow-rate capacities of traps connected to different types of fixtures. These have been summarized in Table 3.5. A typical grease trap used in a commercial application is illustrated in Figure 3.7.

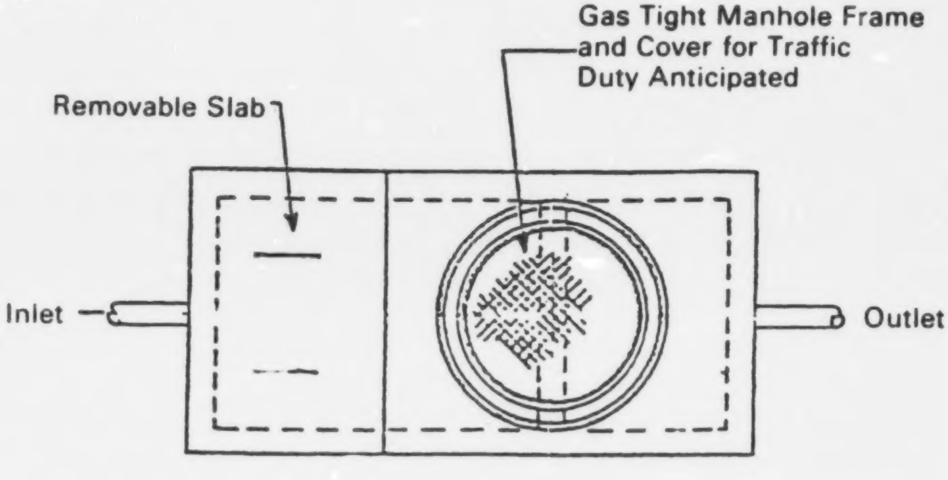
TABLE 3.5

RECOMMENDED RATINGS FOR COMMERCIAL GREASE TRAPS (US PHS, 1974)

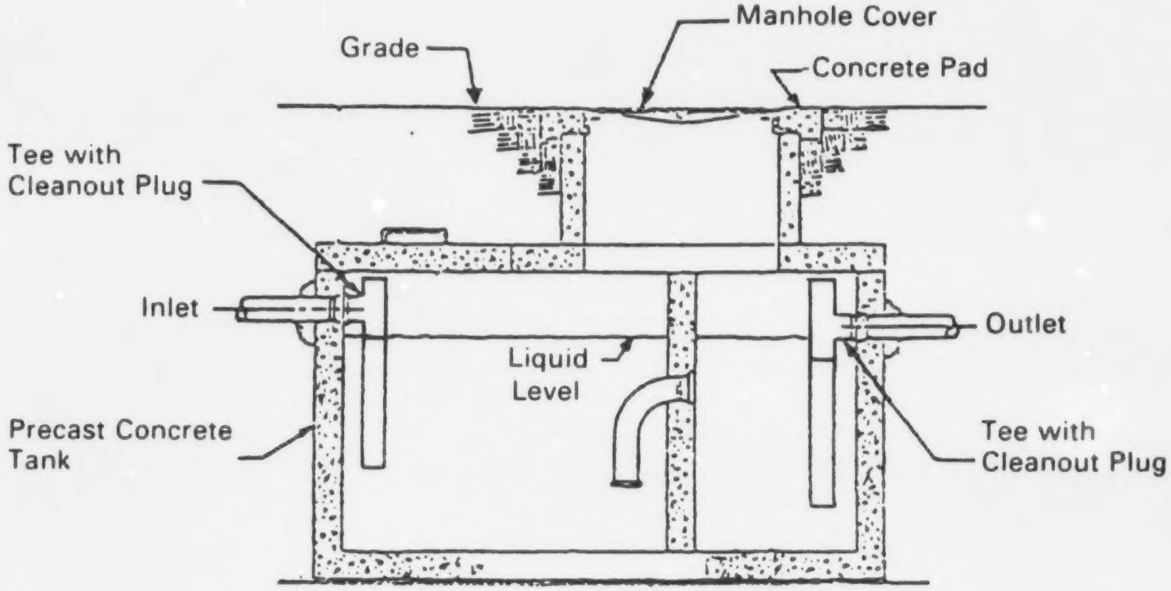
Type of Fixture	Flow Rate (L/min)	Grease Retention Capacity Rating (kg)	Recommended Maximum Capacity Per Fixture Connected to Trap (L)
Restaurant kitchen sink	55	15	190
Single-compartment scullery sink	75	18	190
Double-compartment scullery sink	95	23	235
2 single-compartment sinks	95	23	235
2 double-compartment sinks	130	32	330
Dishwashers for restaurants:			
Up to 110 L water capacity	55	15	190
Up to 190 L water capacity	95	23	235
190 to 380 L water capacity	150	35	380

3.3 SEPTIC TANK OPERATION AND PERFORMANCE FACTORS

One of the major advantages of a septic tank is the complete absence of moving parts and the resulting requirement for minimal maintenance. The septic tank, however, supports a complex biological community whose population fluctuates dramatically depending on the availability of food, the temperature of the microbial environment and the gross chemistry of the contents. During the winter months, tank liquid temperatures are in the range of 4 to 10°C or lower (Hickey and Duncan, 1966). This decline from summer tank temperatures of 18 to 27°C results in a microbial species


Gas Tight Manhole Frame
and Cover for Traffic
Duty Anticipated
Removable Slab
Inlet
Outlet

Top View


Manhole Cover
Grade
Concrete Pad
Tee with
Cleanout Plug
Inlet
Outlet
Liquid
Level
Precast Concrete
Tank
Tee with
Cleanout Plug

Section

Figure 3.7

DOUBLE COMPARTMENT GREASE TRAP (USEPA, 1980) adjustment from a dominantly mesophilic bacteria to cryophilic bacteria and with a corresponding reduction in overall metabolic activity. In order to study these effects, the same authors evaluated the performance of septic tanks at 0.6°C, 4.4°C, and 15°C. Effluent chemical characteristics indicated the highest degree of biological activity at 15°C. However, the BOD and suspended solids removals were similar in all three tanks. BOD removals in all three tanks were approximately 50 percent while suspended solids removals were 100 percent. Hindin et al. (1962) reported similar results.

The US EPA (1980) summarized septic tank effluent quality collected by various researchers. The data are summarized in Table 3.6. The document goes on to state that bacterial concentrations in septic tank effluent are not significantly changed, oil and grease removal is typically 70 to 80% (thereby yielding an effluent with 20 to 25 mg/L oil and grease) and phosphorus removal is only about 15% (yielding an effluent of about 20 mg/L total P).

One cause of septic tank malfunction is failure of owners to pump out the sludge solids when required. As sludge depth increases, the effective liquid volume and retention time decrease accordingly. As this occurs, sludge scouring increases, treatment efficiency falls off and more solids escape through the outlet.

The authors of US EPA (1980) suggested physical inspections of sludge and scum depths to determine when a tank needs pumping. They proposed the following procedures:

1) Scum can be measured with a stick to which a weighted flap has been hinged. As the stick is forced through the mat vertically into the liquid below, the hinged flap falls into a horizontal position. The stick is raised until resistance from the bottom of the scum layer is felt. The scum depth is then measured as depth of submergence of the flap.

TABLE 3.6

SUMMARY OF EFFLUENT DATA FROM VARIOUS SEPTIC TANK STUDIES

Parameter	Ref. (MERL, 1978) 7 Sites	Ref. (Weibel et al; 1949) 10 Tanks	Ref. (Salvato, 1955) 19 Sites	Ref. (Bernhart, 1967) 4 Sites	Ref. (Laak, 1973) 1 Site
BOD_5					
Mean, mg/L	138	138[a]	140	240[b]	120
Range, mg/L	7-480	64-256	--	70-385	30-280
No. of Samples	150	44	51	21	50
COD					
Mean, mg/L	327	--	--	--	200
Range, mg/L	25-780	--	--	--	71-360
No. of Samples	152	--	--	--	50
Suspended Solids					
Mean, mg/L	49	155[a]	101	95[b]	39
Range, mg/L	10-695	43-485	--	48-340	8-270
No. of Samples	148	55	51	18	47
Total Nitrogen					
Mean, mg/L	45	--	36	--	--
Range, mg/L	9-125	--	--	--	--
No. of Samples	99	--	51	--	--

[a] Calculated from the average values from 10 tanks, 6 series of tests.

[b] Calculated on the basis of a log-normal distribution of data.

2) Sludge depth can be measured with the use of a long stick wrapped with rough white towelling and lowered to the bottom of the tank. If left for a few minutes, the location of sludge layer can be distinguished by sludge particles clinging to the towelling.

The depth of scum and sludge should be measured in the vicinity of the outlet baffle. The authors also recommended that the tank should be cleaned whenever one or both of the following conditions occur:

1) the bottom of the scum layer is within 8 cm of the bottom of the outlet device; or
2) the sludge level is within 20 cm of the bottom of the outlet device.

Additional comments related to septic tank operation and performance are as follows (US EPA, 1980):

1) The leaving of sludge solids in the septic tank as an aid to restarting the system is not necessary.

2) The septic tank should not be disinfected, washed or scrubbed when emptied.

3) Special chemicals are not required to start microbial activity.

4) Once tank operation is underway, no chemicals are needed to improve or assist tank operation. Proprietary chemical additives for purposes of "cleaning" of septic tanks may cause sludge bulking and decreased digestion. Other preparations, claimed by manufacturers to eliminate or reduce the need for septic tank pumping are not necessary for proper operation and are of questionable value.

5) Materials not readily decomposed (sanitary napkins, coffee grounds, cooking fats, bones, wet-strength towels, disposable diapers, facial tissues, cigarette butts) should not be flushed into a septic tank since they will not degrade and they can clog inlets, outlets, pumps and disposal fields.

3.4 ALTERNATIVES TO SEPTIC TANKS

Site specific constraints to septic tank effluent disposal sometimes require consideration of alternative treatment systems with less stringent limitations to effluent discharge. Aerobic biological treatment processes can be employed to remove substantial amounts of BOD and suspended solids that are not removed by septic tanks. A secondary feature of these processes is the nitrification of ammonia and reduction of pathogenic organisms. The result is an effluent which has a lower clogging effect on soil systems and which may be acceptable, in some cases, for surface discharge to soil or to a watercourse.

Hutzler et al. (1977) reported that over 60 manufacturers of such units had been identified in the United States. Bailey and Wallman (1971) estimated that approximately 20,000 to 30,000 units had been installed in the United States. The typical aerobic treatment unit is constructed to function in a manner similar to a small extended aeration activated sludge treatment plant. Some have presettling chambers, although many do not. The main treatment compartment, the aeration unit, has a design hydraulic detention time of one or two days. Aeration is generally provided with compressed air and aided by mechanical stirring. There is usually a small final settling tank with ports or slots for gravity sludge return to the aeration chamber. Variations to the aeration chamber concept include the use of trickling filters or rotating biological contactors. Aerobic treatment units may be installed above ground or buried. A typical aerobic treatment unit is illustrated in Figure 3.8.

Several manufacturers have developed sewage treatment units capable of providing an effluent suitable for recycling, usually as flushing water, thereby reducing the quantity of treated effluent requiring disposal to the soil. Selby and Monkes (1979) described

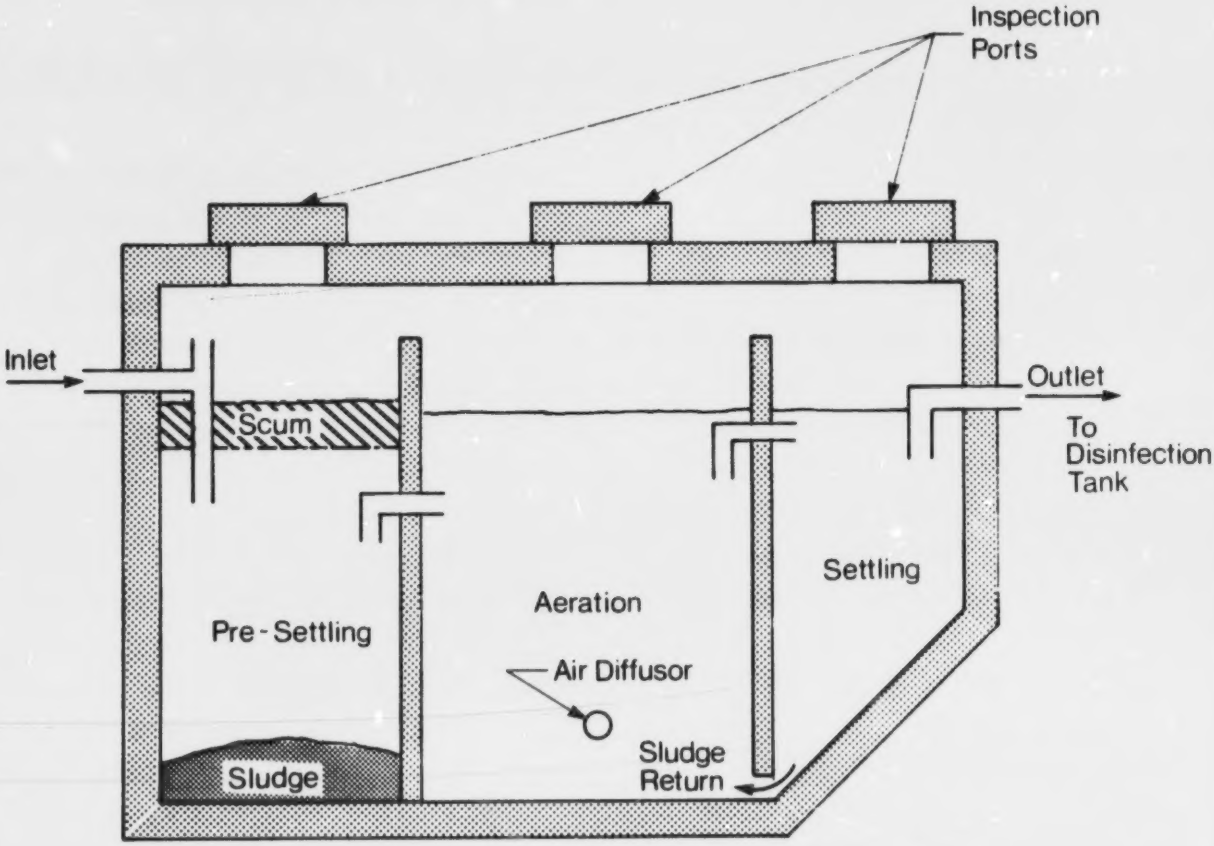

Inspection
Ports
Inlet
Scum
Outlet
To
Disinfection
Tank
Settling
Aeration
Pre - Settling
Air Diffusor
Sludge
Return
Sludge

Figure 3.8

TYPICAL AEROBIC UNIT

the Pure Cycle system, for treatment and recycle of sewage from an individual single-family home. It consists of the following treatment processes operated in sequence:

1) anaerobic digestion in a septic tank;
2) aerobic treatment with a rotating biological contactor;
3) ultrafiltration to remove suspended solids;
4) carbon adsorption to remove dissolved organic compounds,
5) ion exchange resins; and
6) disinfection via ultraviolet radiation.

Jordan (1981) reported the cost-effectiveness of a similar treatment system used to treat the sewage generated by a 5575 m^2 shopping centre in Washington D.C. The inavailability of land for a large soil disposal field prevented the use of a septic tank - drainage field system.

Case history reports of the operation of aerobic treatment units are quite variable. Several favourable reports on their use have been documented by manufacturers and distributors. The papers by Selby and Monkes (1979) and Jordan (1981) have already been mentioned. The major advantage of aerobic systems is their ability to produce an effluent suitable for disposal to surface water or at least into soils where groundwater contamination may be of potential concern. Recycle systems also reduce the quantity of effluent requiring disposal - an asset where soil permeability is low or where groundwater contamination concerns exist.

Other researchers have identified potential problems with aerobic treatment units. Ross et al. (1980) documented some of these problems:

1) Organic and hydraulic shock loads and prolonged power failures tend to upset the quality of effluent discharged.

2) Air supply equipment for aeration and for the operation of air lift pumps is susceptible to plugging.

3) The useful life of filter systems, incorporated in the design of some plants, may be shortened by frequent plant upsets.

Otis and Boyle (1976) evaluated septic tank and aerobic treatment systems in Wisconsin, and concluded that:

1) While aerobic units achieved significantly higher BOD_5 removal than septic tanks (effluent BOD_5 of 47 mg/L vs 158 mg/L), the effluent total suspended solids concentrations were similar (53 and 54 mg/L).

2) Septic tanks were more stable than aerobic units with less variability in effluent quality for most parameters monitored.

3) Near complete nitrification was achieved in aerobic units while ammonia and organic nitrogen were the dominant forms in septic tank effluent.

They recommended the following design and maintenance requirements for aerobic units:

1) a septic tank or trash trap preceding the unit to remove grease, floating solids and large debris;
2) some form of flow equalization, either with a surge tank ahead of the unit or by constant rate pumping of mixed liquor from the aeration chamber to the clarifier;
3) a positive sludge return operating on a regular cycle of at least twice daily;
4) a heat source to maintain mixed liquor temperatures above 15°C during the winter;
5) effluent weirs at least 610 mm in length for units servicing single homes;
6) scum and deflection baffles in the clarifier;
7) regular and competent maintenance including inspections at least once every two months; and
8) sludge removal every eight to twelve months.

Voell and Vance (1974) surveyed 151 aerobic treatment units over a two year period and observed the following:

1) 10 percent of the units had control problems;
2) 6 percent had problems with blowing electrical fuses;
3) odours were noted with 10 percent;
4) 75 percent of the dry-feed chlorinators were empty; and
5) 57 percent lost solids into the effluent.

Glasser (1975) monitored aerobic treatment units of five different manufacturers and recommended regular maintenance service at least four times per year to ensure satisfactory and consistent performance.

Hutzler et al. (1977) reported the following from their 18 month survey of 36 operating aerobic treatment units:

1) the average life span of air blowers was only two years;
2) liquid chlorination pumping equipment usually plugged or lost its prime;
3) the high humidity found inside the tank tended to shorten the life of mechanical components;
4) electrical controls were too complicated;
5) improper installation (some installed backwards, others completely buried and some installed in low areas where they were subject to flooding);
6) improper insulation resulted in freezing of chlorine contact tanks; and
7) aeration equipment was often inoperative because of blown fuses or deliberate inactivation by the home owners to save operating costs.

Ross et al. (1980) compared septic tanks and aerobic plants on the basis of the following parameters:

1) their intended function;
2) their performance;
3) operation; and
4) cost.

They explained that the function of a septic tank is to separate solid from wastewater by sedimentation and flotation in addition to stabilizing pollutants. Aerobic units stabilize wastewater by converting organic matter into microbial cells, carbon dioxide and water. The resultant clarified wastewater is very amenable to soil renovation and removal processes.

The same authors concluded from their literature review that BOD and suspended solids concentrations from septic tanks are about 150 to 200 mg/L, whereas values for aerobic units can be expected to be 60 to 100 mg/L. In addition, a dissolved oxygen concentration of about 1 to 3 mg/L can be expected in the effluent from aerobic units.

They recommended that septic tanks should be inspected frequently (three times) during the first year of operation and once annually thereafter. The following inspection programs for aerobic units were recommended:

1) once daily by the home owner during the first few weeks and once every two or three months thereafter;
2) semi-annual or annual inspections by qualified technicians; and
3) intermittent removal of sludge and scum approximately every 9 to 12 months.

Finally, they compared the capital and operating costs for the two processes. The capital cost of septic tank installation was estimated to be 10 cents to 15 cents per litre with an operating cost of 10 to 15 dollars per year per tank. Aerobic units were estimated to cost between \$1.10 to \$2.20 per litre installed; maintenance contracts varied from \$65 to \$110 per year and power costs ran from \$35 to \$60 per year. Effluent disposal restrictions, rather than cost, generally dictate the selection of options to the conventional septic tank.

SECTION 4

DRAINAGE FIELD DESIGN PRACTICES

4.1 SITE SUITABILITY EVALUATIONS

4.1.1 Introduction

The effectiveness and acceptability of septic tank - drainage field systems for sewage treatment and disposal depends largely upon the soil's capability for absorbing and purifying the partially treated wastewater. The major factors which affect the suitability of septic tank - drainage field systems include:

1) type of soil material characteristics including texture, structure, stratification, bulk denstiy, grain size distribution, and permeability rate;
2) depth to bedrock and saturated soil;
3) presence of bedrock fissures or fractures;
4) surface slope; and
5) the presence of ponding and flooding conditions.

Several procedures to evaluate site suitability have been presented in the literature and these findings and recommendations are discussed in this section.

4.1.2 The Soil Environment and its Treatment Mechanisms

The soil in a disposal field does more than receive and dispose of the wastewater. It is a complex physical, chemical and biological environment providing the resources necessary for wastewater renovation. Soil bacteria break down the soluble organic pollutants. The physical filtering action of the soil itself removes suspended particles including sewage borne bacteria and viruses. The soil particles can also react chemically with the applied effluent. Soil particles which are negatively charged act as cation exchangers thereby retaining certain positively charged substances.

The factors influencing soil treatment are a complex interaction of biological, physical and chemical processes, and the characteristics of the soil environment greatly affect the processes and influence these inter-relationships. Otis et al. (1977) described how the prolific growth of soil bacteria in a disposal field causes the production of slimy films of bacteria on the soil particles. These slimes, along with the fine pore system of the soil act as a trap or filter for suspended solids and additional bacteria, thus creating a crust or clogging zone. The clogging in this zone may be enhanced under some conditions by swelling of clay sized soil particles and subsequent alteration of the soil fabric.

Cotteral and Norris (1969) explained why the formation of the crust or clogging zone is an important mechanism in the operation of disposal fields. In naturally well-drained soils such as sands and loams, since the crust impedes the flow of water into the soil horizons below the bed, they will be unsaturated. Tyler et al. (1977) estimated that the unsaturated flow rate through a sandy soil may be more than 100 times less than the saturated rate of flow. This reduced rate of flow allows the effluent to be in contact with the soil a considerable length of time to maximize treatment efficiency.

The flow rate into the unsaturated soil zone is a function of the head of effluent above the crust, the resistance of the crust, and the moisture potential of the underlying soil. Sikora and Corey (1975) described how different crust-soil, wastewater loading and ground water combinations resulted in differing degrees of soil saturation and aeration. Based on their knowledge of measured soil moisture conditions in various soils, they estimated that soils with sandy, sandy loam and loamy textures would be generally unsaturated and aerobic below the disposal field, while clay soils would be nearly saturated and anaerobic. They explained that in extreme cases of saturation the crust-soil system will not accept all the added effluent and some will "back-up" and spill over the surface of the ground. Methods of rejuvenating clogged disposal beds are discussed in Section 4.5.

Tyler et al. (1977) outlined the relationship between soil permeability and soil treatment efficiency. Septic tank effluent passing through the crust layer and going slowly through a soil will have time for many reactions to occur at the surfaces of particles and in the soil solution in contact with air-filled voids. If, however, flow is very rapid as in a gravelly soil or through large cracks in bedrock, very little time is available for purification to occur. It is also possible to have channels or cracks in fine textured soils, usually as the result of natural breaks between soil structural units. These channels adjacent to a region of saturation will result in continuous saturated flow or short circuiting through the soil pores.

The same authors explained in detail the soil interactions involved in renovation of wastewater in disposal beds. Pathogenic bacteria and viruses are usually removed through adsorption on charged soil particles, by filtration or by die-off due to attrition of nutrients. Others are removed by the action of toxic chemicals in the environment or by the accumulated end products of the bacteria themselves. Nitrogen is removed by adsorption volatilization, biological uptake by soil bacteria, and by the biologically mediated processes of nitrification followed by denitrification. Phosphorus is initially adsorbed on charged soil particles through ion exchange reactions, and then may later form chemical precipitates.

The ability of the soil to purify septic tank effluents is greatly influenced by the nature of flow through the soil crust interface and the presence of an unsaturated zone below the disposal field. The removal of wastewater constituents, particularly nitrogen and phosphorus, has been the subject of numerous studies not specifically discussed in this report.

4.1.3 Site Suitability Factors

Authors such as Baker and Bouma (1975) and Healy and Laak (1974) described the factors which must be considered when evaluating the suitability of a site for a septic tank - disposal field system. These included:

1) the size and slope of the lot;
2) the location in relation to streams, wells and structures;

3) the character and extent of the soil mantle; and
4) the depth to the groundwater table.

Many of these assessment criteria have been incorporated into regionally specific design codes. For example, the Central Mortgage and Housing Corporation (CMHC) Standards suggest that where septic tank - disposal field systems are to be used, the lot sizing criteria should take into account the site slopes as well as areas unsuitable for disposal fields because of local variations in soil conditions. They also suggest that provision should be made for future replacement of disposal fields. They have defined the tank sizing and have specified minimum distance location of disposal fields from buildings, trees, wells and streams. The specific standards applied by various regulatory agencies will be discussed in more detail in Section 6.

The suitability of the soil for absorbing wastewater and the absorption area required are determined by various soil characteristics. Cotteral and Norris (1969) proposed that soil borings should be used to establish the depth, extent and character of the soil mantle, and the location of bedrock or other impervious strata. They explained that this investigation should reasonably define the site characteristics within the upper 1.5 metres and involve as many borings as necessary to cover the drainfield area. Investigation should also confirm that the soil possesses sufficient percolative capacity to be able to support the leaching process. Specific methods of performing these analyses are discussed in Section 4.4.

Parker et al. (1977) suggested the following soil profile evaluations for suitability of disposal fields:

1) Soil Colours and Colour Patterns
Bright colours indicate well-drained, well aerated conditions, whereas dull and grey colours usually indicate poorly aerated wet subsoils. Layers subject to alternating periods of saturation are generally mottled with brown, yellow-orange and grey colours.

2) Layering of Textures
This restricts vertical percolation and may cause mottled soil colours.

3) Texture and Structure

This controls the rate of vertical percolation.

4) Bedrock Conditions

The physical and chemical composition of bedrock controls the rate at which wastewater can pass through and the degree of treatment which can occur.

They suggested that the following methods may be used to evaluate the soil's ability to absorb and transmit effluent:

1) determination of soil density;
2) determination of porosity and pore size distribution;
3) examination of plant root distribution;
4) identification of restrictive layers such as claypans and fragipans; and
5) measurement of hydraulic conductivity, permeability and percolation rates.

Site specific evaluation procedures are particularly important where septic tank - disposal field systems are used for small communities, multi-unit development or commercial and industrial applications. Otis (1982a) suggested that, for these larger systems, site suitability evaluations should begin with visual surveys which provide the following information:

1) general site features including topography, rock outcrops, buildings and roads;
2) landscape positions relative to land forms and topography;
3) area of flooding hazard;
4) extent of surface vegetation;
5) land slopes; and
6) horizontal setbacks.

The author also stressed the use of soil borings to define soil suitability criteria. He suggested an excavated pit large enough to enter, although a hand auger or probe was deemed suitable if the evaluation was conducted by an experienced soil scientist. He also suggested the digging of pits around the perimeter of the area with sufficient pits or borings made to describe the following:

1) soil horizons;
2) texture within each horizon;
3) soil structure within each horizon;
4) colour;
5) bedrock depth;
6) soil bulk density; and
7) presence of clay layers.

In the same study, the author discussed hydraulic conductivity testing as the second stage of investigation. The most common method used is the "percolation test". The accuracy of this test and alternative methods of assessing hydraulic conductivity are covered in Section 4.4.

If the disposal field is to be used for disposal of large quantities of wastewater, Otis (1982a) stressed a necessity for the following hydrogeologic investigations:

1) groundwater elevation including seasonal variations;
2) horizontal groundwater gradients; and
3) hydraulic conductivitiy of the soil within the saturated zone.

Gayle et al. (1981) selected a study area in North Carolina which had a large number of homes on small lots, each having its own septic tank - disposal field system. Septic tank failures were readily apparent in the study area. The fields were boggy due to surfacing effluent, with an accompanying foul smell, and significant fly populations were present. The area's water supply came from individual wells. The authors developed a procedure of investigations for designing disposal field systems for problem sites. Problem soils were defined as those having soils of very low permeability, steep slopes above 14 percent, high water table, shallow depth to impermeable layer, and short distances to streams, lakes, and rivers. The procedures used by the authors and their observed results are summarized in Table 4.1.

TABLE 4.1
PROCEDURES FOR DESIGN OF DISPOSAL FIELDS ON PROBLEM SITES AS ILLUSTRATED BY A NORTH CAROLINA STUDY (Godfrey et al. 1981)

Design Procedures	Methods Used	North Carolina Results
1) Detailed topographic survey	1) grid method - 30 m intervals	1) slopes of 0 to 14 percent
2) Soil-type evaluation	1) on-site survey with the use of pits (1.5 to 3.0 m deep) on a 60 x 45 m grid 2) auger holes at close intervals	1) marked changes in soil types, soil structure and groundwater elevations were observed even within distances of a metre or less
3) Hydraulic conductivity measurements	1) auger hole method 2) pit method 3) double-ditch method	1) good correlation between procedures used 2) marked variability across the site (at least 50 percent with all methods used)
4) Percolation test	1) standard test defined by the Guilford County Board of Health	1) a range of 2.3 to 3.1 cm/hr
5) Depth to impermeable layer	1) observation pits 2) auger holes	1) restrictive layer of clay and saprolite found at depth of 60 to 100 cm
6) Depth to groundwater	1) auger holes 2) observation pits (1.5 to 3.0 deep) 3) wells equipped with water-level recorders	1) high water table periodically (25 to 30 cm from the surface) when compared with adjacent water tables in sewered areas
7) Water quality studies	1) shallow wells (3 m) 2) deep wells (12 m) 3) monitoring of domestic wells	1) considerable fecal coliform contamination observed in deep and shallow wells 2) nitrates and phosphorus were below recommended limits 3) heavy metals were negligible

4.2 DOSING SYSTEM ALTERNATIVES

4.2.1 Methods of Wastewater Application

The selection of a particular method for application of effluent to a disposal field is an important design component. The application system must be compatible with the local soil and site characteristics. Otis et al. (1977) summarized the basic methods of application as follows:

1) continuous ponding - maintaining a head of wastewater above the infiltrated surface;
2) dosing and resting - a period of loading followed by a resting phase to allow soil drainage and reaeration to occur; and
3) uniform application without ponding at a rate lower than that at which the soil can accept liquid.

The same authors suggested that the selection of application method is a function of soil texture. They stated that uniform application without ponding is the best loading method for most soil and site conditions and that it is an essential method during initial operation of disposal fields in rapidly permeable soils when no clogging layer is present. In fine textured soils, however, it is not always possible to maintain a loading regime of uniform application without ponding due to excessive clogging. The authors suggested that continuous ponding is often necessary in these soils to provide the necessary hydraulic gradient across the clogging layer. Bouma et al. (1975) stressed that dosing and resting should not be used in highly permeable soils with a high water table unless small, frequent doses are applied each day. Long periods of aeration would permit the clogging mat to degrade, thus allowing saturated flow below the disposal bed.

Otis et al. (1977) summarized the physical site conditions other than soil texture which may dictate the method of application which should be used. These included the presence of a high groundwater table, shallow impermeable bedrock or a cemented pan. These all require loading methods that spread the wastewater over a large area.

They suggested that uniform application without ponding would be best, although dosing and resting may be suitable if the dosing volume necessary to pond the infiltrative surface is not excessive. The authors stressed that on steeply sloping sites, uniform application without ponding would be most appropriate to prevent downslope seepage.

Various methods are used today to attempt to achieve uniform application of the liquid over the entire infiltrative surface. The designs used today include large diameter perforated pipe networks, pressure distribution networks and other proprietary designs.

4.2.2 Large Diameter Perforated Pipe Networks

Otis et al. (1977) stated that the most common distribution system in use today consists of a plastic pipe, 10 cm in diameter and having two rows of holes near the invert 45° off vertical centre. The holes are 13 to 16 mm in diameter and spaced 76 mm apart. The authors summarized the system as follows:

1) the pipe is laid level or on a 0.17 to 0.33 percent slope;
2) effluent flows into the pipe by gravity;
3) the number of pipes used and the distance between them is a function of the type of disposal field design; and
4) the pipes are interconnected by a common solid header pipe, drop box or distribution box.

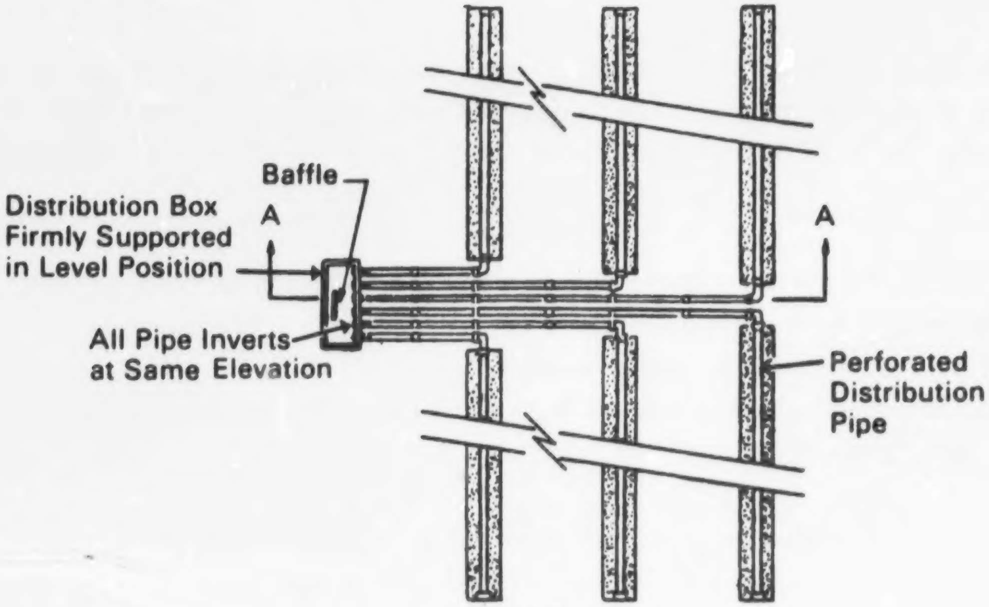
The purpose of laying the pipe on a true, prescribed slope and a prescribed distance apart is to get uniform distribution as the effluent trickles or flows in by gravity. McGauhey and Winneberger (1965) and Bouma et al. (1972) suggested that this rarely occurs in actual practice. They observed that effluent seems to exit out of a few holes either at the inlet area, middle, or far end of the pipe. This causes localized overloading since small areas receive a continuous trickle of effluent, thereby promoting saturated flow conditions. As a result, clogging eventually occurs at that location and reduces infiltration below the rate at which effluent is discharged. Effluent is forced to flow further along the pipe until it reaches an unclogged area.

This phenomenon known as "creeping failure" continues until the total bottom area of the disposal field is clogged. The authors reported that complete disposal field failure could be prevented, however, if the system were sized such that the loading rate was less than or equal to the infiltration rate through the clogged soil.

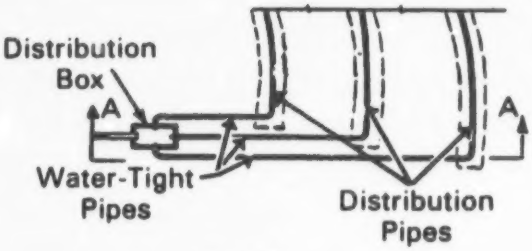
Otis et al. (1977) concluded that the function of the perforated pipe is simply to convey the effluent to the trench or bed. Laying the pipe at a prescribed uniform slope and spacing is of no value because the effluent will exit the hole of lowest elevation. Dosing does not greatly improve distribution because of the large number of holes. They suggested that large diameter perforated pipe networks are best suited for continuously ponding or dosing and resting loading regimes. Requirements for uniform slopes and specific spacings of the pipe are unwarranted. They suggested that the critical factor for dosing applications is the discharge of a sufficiently large volume of liquid with each dose to submerge the entire infiltrative surface.

When more than one distribution pipe is required, some means of dividing the flow between pipes is necessary. Otis et al. (1977) reported that the systems consist of interconnection of pipes with the use of a common solid header pipe or interconnecting closed loops. Both were reported suitable where the entire infiltrative surface is at one elevation, such as in beds or multi-trench systems or level or nearly level sites.

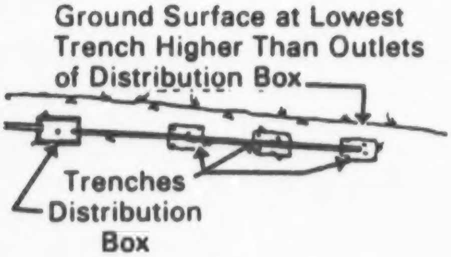
Several authors have discussed the use of a distribution box (illustrated in Figure 4.1). The US EPA (1980) described it as a network of laterals extending from a common watertight box. The box may be round or rectangular, with a single inlet, and an outlet for each distribution lateral. It has an exposed, removable cover. Its purpose is to divide the incoming wastewater equally between each lateral. The authors stated that in order to achieve this objective, the outlet inverts must be at exactly the same elevation and the inlet invert should be 25 mm above the outlet inverts. Where dosing is employed or where the slope of the inlet pipe imparts a significant velocity to the wastewater flow, a baffle should be placed in front of the inlet to prevent short-circuiting.


Baffle
Distribution Box
Firmly Supported
in Level Position
A
A
All Pipe Inverts
at Same Elevation
Perforated
Distribution
Pipe

Central Feed


Distribution
Box
A
A
Water-Tight
Pipes
Distribution
Pipes

End Feed


Ground Surface at Lowest
Trench Higher Than Outlets
of Distribution Box
Trenches
Distribution
Box

Section A-A

Figure 4.1

DISTRIBUTION BOX NETWORK (USEPA, 1980)

Coulter and Bendixen (1958) concluded that distribution boxes were ineffective because in practice, the outlet inverts are rarely at the same elevation. Either poor construction or differential settling during construction create unequal distribution.

Machmeier (1979) suggested that distribution box networks be used only for absorption systems located on level or gently sloping sites, where the system can be installed so that the ground surface elevation above the lowest trench is above the box outlets. This would ensure that in an extreme case of uneven distribution, effluent would not surface above flooded laterals. It would instead back up into the distribution box and flow into a different lateral.

Otis et al. (1977) described the drop box or serial distribution method (illustrated in Figure 4.2) as being particularly suited for sloping disposal sites. It consists of a network that serially loads each trench to its full hydraulic capacity. The USEPA (1980) described the drop box itself as a circular or square box with a removable cover. It has an inlet, one or two distribution lateral outlets, and an overflow. The lateral outlet inverts are located at or near the bottom of the box and are all of the same diameter. The authors recommend the following:

1) that the overflow invert be the same elevation as the crown of the lateral outlet, or up to 50 mm above it, in order to cause the full depth of the trench to flood;
2) that the inlet invert of the drop box be at the same elevation as the overflow invert or a few centimetres above;
3) that an elevation difference of 25 to 50 mm between trenches is all that is needed to install a drop box network; and
4) that the boxes be buried, although they suggested that the covers be left exposed for periodic inspection and maintenance.

The same authors explained that drop boxes are installed at the wastewater inlet of each trench with these inlets being located anywhere along the trench length. A solid wall pipe connects the overflow from the higher box to the inlet of the lower box. They described the sequence of operation. The first box in the network receives all the effluent from the pretreatment tank and distributes it into the first trench. When

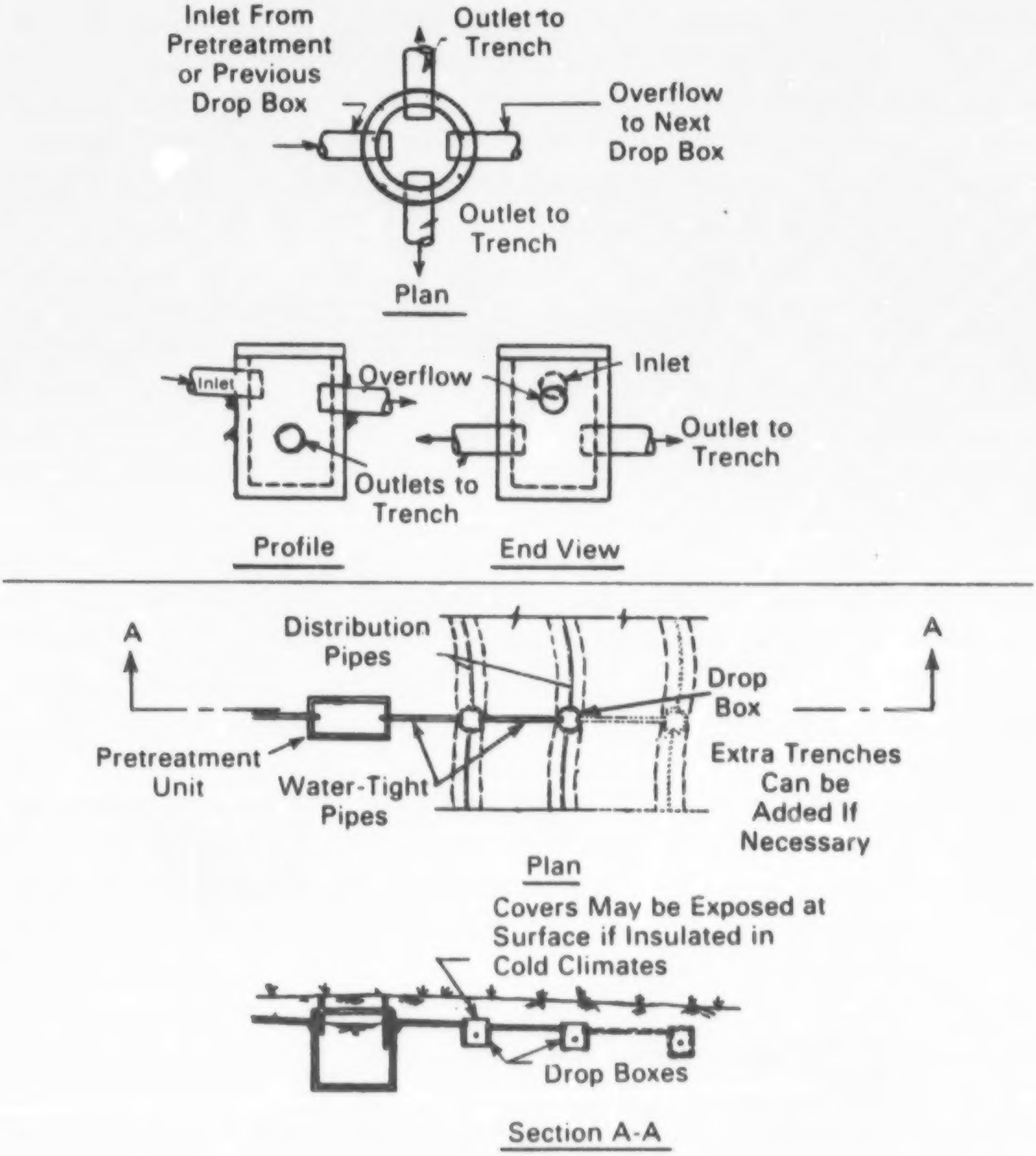

Inlet From Pretreatment or Previous Drop Box
Outlet to Trench
Overflow to Next Drop Box
Outlet to Trench
Plan
Inlet
Overflow
Outlets to Trench
Inlet
Outlet to Trench
Profile
End View
A
Distribution Pipes
A
Drop Box
Pretreatment Unit
Water-Tight Pipes
Extra Trenches Can be Added If Necessary
Plan
Covers May be Exposed at Surface if Insulated in Cold Climates
Drop Boxes
Section A-A

Figure 4.2

DROP BOX DISTRIBUTION
NETWORK (USEPA, 1980)

the first trench fills, the box overflows into the next trench. In this manner, each trench in the system is used successively to its full capacity. Thus, only the portion of the system required to absorb the wastewater is used. During periods of high flow or low absorptive capacity of the soil, more trenches will be used. When flows are low or during the hot dry summer months, the lower trenches may not be needed, so they may drain and dry out, automatically resting more trenches, hereby rejuvenating their infiltrative surfaces.

Otis et al. (1977) suggested that drop box distribution methods may result in more severe clogging of the first trench in the system because of the heavier solids load and deeper ponding in the trench. They suggested that this problem can be alleviated if the drop boxes are equipped with valves to allow any trench in the system to be shut off for resting and rejuvenation. They recommended against the use of drop box systems in highly permeable soils.

The US EPA (1980) described the serial distribution method (Figure 4.3), also referred to as relief line networks, as an alternative to the drop box method. It uses overflow or relief lines between trenches in place of drop boxes. The authors made the following design recommendations:

1) the invert of the overflow section should be located near the top of the porous media in order to use the maximum capacity of the trench;
2) where a gravity feed system is used, the invert of the overflow from the first absorption trench should be at least 100 mm lower than the invert of the septic tank outlet; and
3) although relief lines may be located anywhere along the trench, successive trenches should be separated by 1.3 to 3.0 m to prevent short-circuiting.

These same authors stated that a major advantage of this system is its usefulness on highly sloping terrain. Disadvantages cited were the difficulty of adding trenches to the system once it is in place and inability to allow resting of the upper trenches.

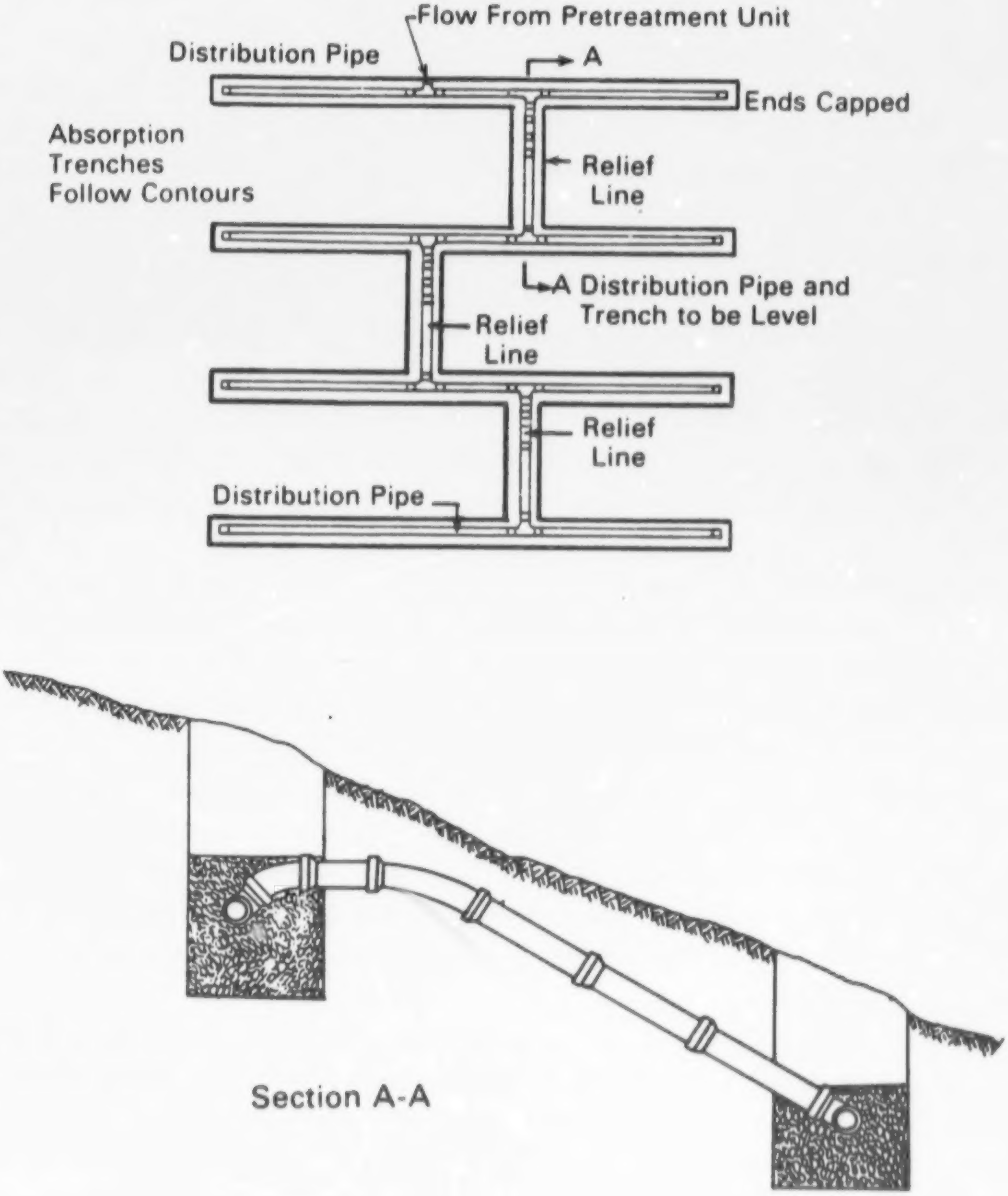

Flow From Pretreatment Unit
Distribution Pipe
A
Ends Capped
Absorption Trenches Follow Contours
Relief Line
A Distribution Pipe and Trench to be Level
Relief Line
Relief Line
Distribution Pipe
Section A-A

Figure 4.3

RELIEF LINE DISTRIBUTION NETWORK (USEPA, 1980)

4.2.3 Pressure Distribution Networks

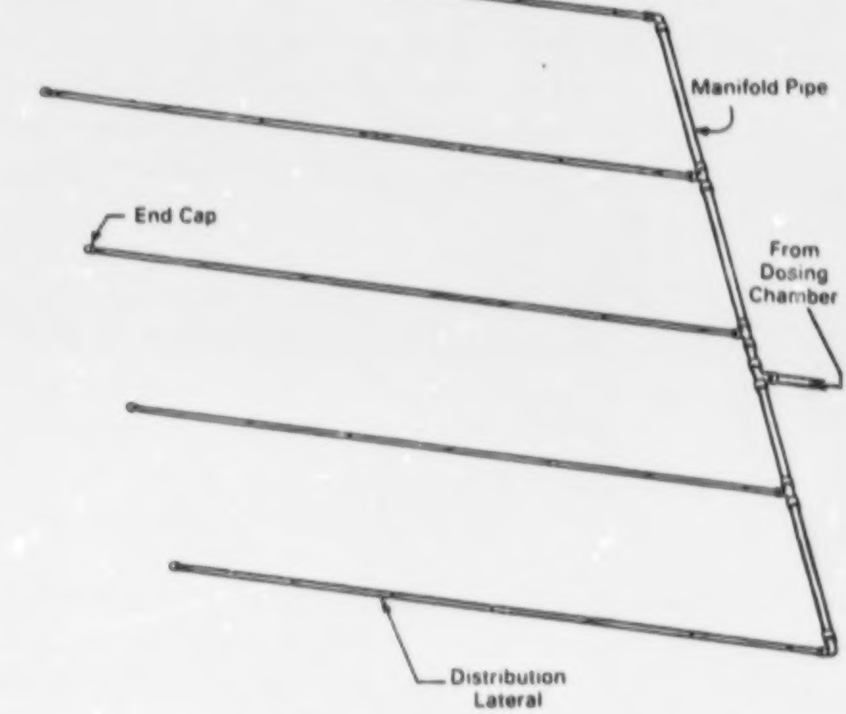
Otis et al. (1977) and the US EPA (1980) reported that pressure distribution networks are utilized where uniform distribution of effluent is required over the entire infiltrative surface. The advantage cited was much better control of application rates and thus, the prevention of local saturated conditions. This is done by putting the network under pressure and sizing the pipe and hole diameters to balance the headlosses to each hole. Figures 4.4 and 4.5 illustrate the components of a pressure distribution network.

Otis et al. (1977) summarized the "rules of thumb" for design of pressure distribution systems as follows:

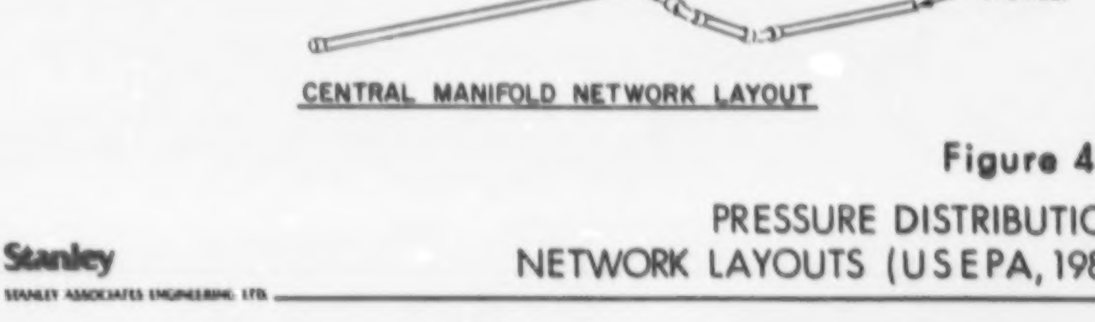
1) assume at least 60 to 90 cm of head at the terminal end of each lateral;
2) assume that 65 to 85 percent of the total headloss in the network occurs across the orifices;
3) assume that 10 to 15 percent of the total headloss occurs in delivering the liquid to each hole; and
4) the remaining 0 to 25 percent of the headlosses occur through fittings.

They also suggested that to ensure uniform distribution, the total dosing time should be 8 to 10 times longer than that necessary to fill the pressure network at the operating discharge rate. They presented a rational design procedure for pressure distribution networks together with a design example. Cogger et al. (1982) have provided a detailed discussion of the procedures and materials to be used for successful siting, design, installation and maintenance of pressure distribution systems.

Converse (1974) demonstrated through experiments that the distribution provided by pressure networks was much more uniform than that provided by large diameter perforated pipe network systems. Converse et al. (1974) reported field evaluations of six pressure distribution systems. The major problem observed was the undersizing of the pump. Hole plugging did not occur with properly sized pumps. Otis (1976) reported the design and installation of a pressure distribution system serving a community in Wisconsin. Because of the large size of disposal field required and the


Manifold Pipe
End Cap
From Dosing Chamber
Distribution Lateral

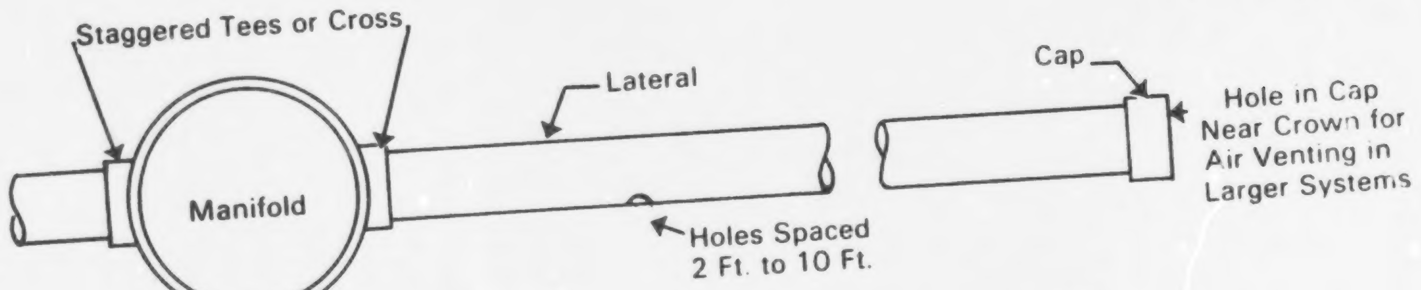
END MANIFOLD NETWORK LAYOUT


Distribution Lateral
Manifold
End Cap
From Dosing Chamber

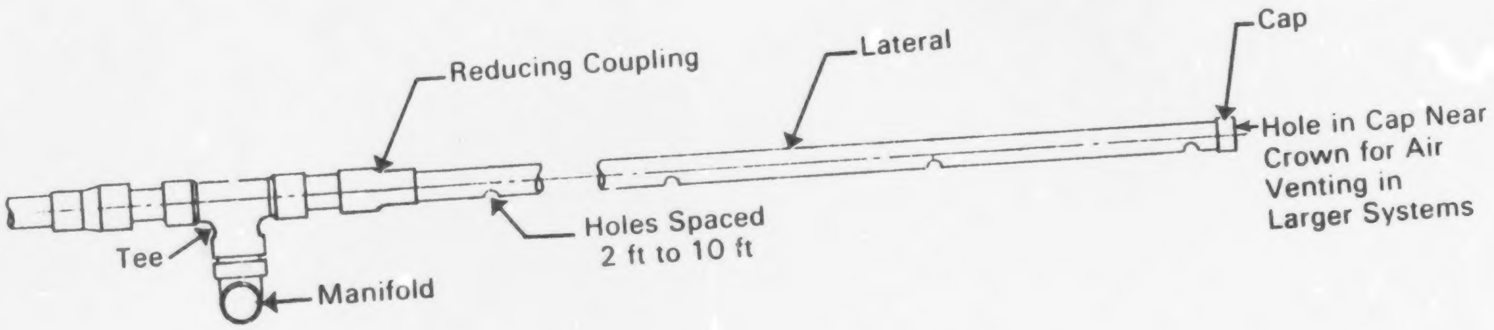
CENTRAL MANIFOLD NETWORK LAYOUT

Figure 4.4

PRESSURE DISTRIBUTION NETWORK LAYOUTS (USEPA, 1980)


Staggered Tees or Cross
Lateral
Cap
Hole in Cap Near Crown for Air Venting in Larger Systems
Manifold
Holes Spaced 2 Ft. to 10 Ft.

STAGGERED TEES OR CROSS CONSTRUCTION


Cap
Lateral
Reducing Coupling
Hole in Cap Near Crown for Air Venting in Larger Systems
Holes Spaced 2 ft to 10 ft
Tee
Manifold

TEE TO TEE CONSTRUCTION

Figure 4.5

PRESSURE DISTRIBUTION LATERAL DETAILS (USEPA, 1980) need to prevent groundwater contamination, a pressure distribution network was constructed. Field tests on the system showed that distribution was uniform and no ponding occurred. Otis et al. (1977) concluded that pressure distribution networks may be the only alternative in areas having highly permeable soils and where groundwater contamination is possible. Carlile (1979) elaborated further by reporting that pressure distribution systems are suitable for the following conditions that prohibit the use of conventional large diameter pipe distribution networks:

1) highly permeable sand;
2) shallow groundwater tables;
3) sites with restrictive horizons at shallow depths;
4) steeply sloped sites; and
5) large wastewater flows.

4.2.4 Other Dosing Techniques

Several other distribution systems have been developed in recent years. Otis et al. (1977) reported the development of the Case System, the panel system, the Ameration Chamber and the Fuldos System. They stated that the Case System, developed in North Carolina, has been promoted for use in slowly permeable soils. It distributes septic tank effluent through a series of porous cement blocks. The blocks are laid end to end in a trench, cemented together for a distance of 12 to 18 m and covered with earth backfill. No gravel is used in the trench. Effluent moves in the hollow chambers of the blocks until it diffuses through the porous block walls into the surrounding soil. The authors concluded that the system will not work where the soil remains wet much of the time or has very low hydraulic conductivity.

They reported a significant shortcoming to the Case system. Although it attempts to achieve uniform application, failures occur if the block does not drain between doses. Therefore, the system must be carefully sized and constructed to insure complete absorption of effluent between doses. This is a difficult problem since the natural soil must be compacted around the block. Excessive compaction or puddling can result, thus significantly changing the hydraulic conductivity of the surrounding soil.

The prefabricated panel system, also developed in North Carolina, is similar to the Case System. It is composed of porous cement panels, each of which is a network of vertically spaced chambers with interconnected tubes at one end. The top wall of the panel has a removable cover for access and inspection of the interior. The panels are placed in a trench and a bed of sand is laid in the bottom of the trench for the panel to rest on, and is packed between the sidewall of the panel and the trench. Effluent enters the top chamber of the panel and diffuses through the bottom and sidewall. As this top chamber fills with effluent, an overflow tube passes the liquid into lower chambers. Otis et al. (1977) concluded that the panel system appeared to be capable of providing uniform distribution of effluent over the full length of the panel thus minimizing the clogging potential at the soil's infiltrative surface.

Otis et al. (1977) also described the Ameration System as consisting of concrete chambers with open bottoms that interlock to form an underground cavern over the exposed infiltrative surface. No pipe or stone is used. The septic tank effluent is discharged into the cavern through a central weir, trough, or splash plate and allowed to flow over the surface in any direction. Vents provide a free flow of air directly to the soil surface between doses. Manholes in the roof of the chamber allow visual inspection of the soil surface and access for necessary maintenance.

They reported two advantages over conventional distribution systems:

1) the infiltrative surface is more readily exposed to air; and
2) access to the surface is provided.

The authors concluded, however, that in soil other than very coarse textured soils, there would seem to be the danger of severe clogging from the migration of fine particles during dosing.

Otis et al. (1977) also described the Fuldos System as a concrete chamber placed on a bed of stone and backfilled with stone. Effluent is dosed into the stone fill outside the chamber at such a rate that it will completely fill the void spaces in the stone. Any excess liquid flows through an inlet at the top of the concrete chamber and is stored until the liquid level drops in the trench. A oneway release outlet at the bottom of the storage chamber allows the liquid to flow out of storage as the level drops in the trench.

They stated that the system is designed to maximize sidewall absorption under a dosing and resting loading regime. They found, however, that its cost was about four to five times higher than conventional systems and it was, therefore, not feasible for individual home owners. They reported that it was most suited for larger operations, especially where flow is quite variable and the storage chamber can thus be better utilized. Otis (1985) reported that because of the disadvantages described, the Case, panel, Ameration Chamber and Fuldos systems are rarely, if ever, used anymore.

The US EPA (1980) reported the use of the inverted distribution network in which the holes in the perforated pipe are located in the crown rather than near the invert. The arrangement has been designed to provide more uniform distribution of wastewater over a large area and to prolong the life of the field by collecting in the bottom of the pipe any settleable solids passing out of the septic tank. Watertight sumps are located at both ends of each inverted line to facilitate removal of the accumulated solids.

Anderson et al. (1983) reported the successful use, in Minnesota, of gravelless seepage trenches. This system uses large diameter (200 mm and 255 mm) corrugated polyethylene tubing covered with a permeable nylon fabric. Two rows of holes, approximately 13 mm in diameter are located at the four o'clock and eight o'clock positions on the tubing. No gravel or rock is placed in the trench or bed since the area of nylon fabric in contact with the soil provides the infiltrative surface. The system is illustrated in Figure 4.6. Sylvester (1985) reported that he had installed 22 such systems in Alberta during the past four years with the largest designed to handle 9000 litres per day of sewage.

4.2.4 Materials of Construction

The US EPA (1980) summarized the materials information for large diameter pipe networks as follows:

1) 8 to 10 cm diameter perforated pipe or drain tile are typically used; and

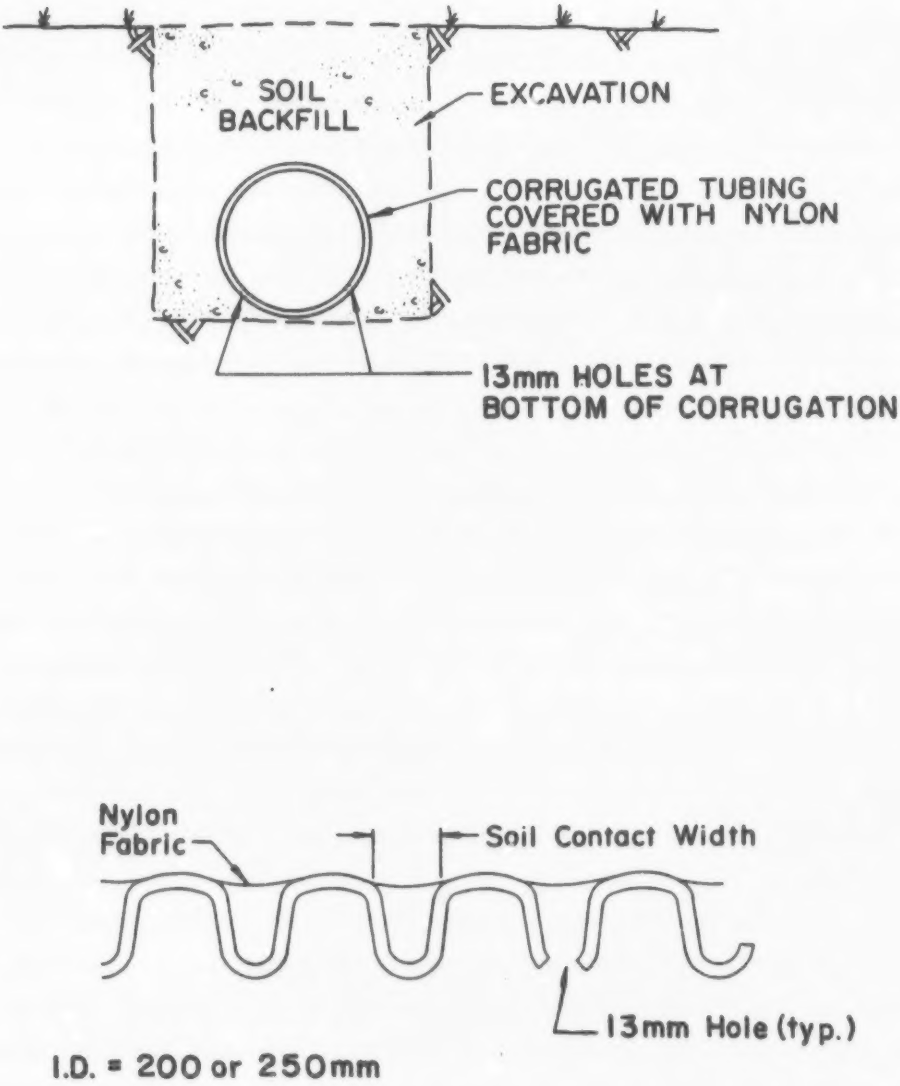

SOIL BACKFILL
EXCAVATION
CORRUGATED TUBING COVERED WITH NYLON FABRIC
13mm HOLES AT BOTTOM OF CORRUGATION
Nylon Fabric
Soil Contact Width
13mm Hole (typ.)
I.D. = 200 or 250mm

Figure 4.6
THE GRAVELLESS SYSTEM (ANDERSON et al, 1983)

2) the perforated pipe commonly has one or more rows of 10 to 20 mm diameter holes, although hole spacing is not critical.

The same publication provided a table of material classes and ASTM specifications for various pipes in use today.

The same authors stated that because of ease of drilling and assembly, plastic pipe is used for pressure distribution networks. These networks are usually constructed on-site.

4.2.5 Flow Diversion for Alternating Disposal Field

For large disposal field systems it is often desirable or necessary to construct several fields and alternate them to provide resting periods for rejuvenation and frost protection. The US EPA (1980) illustrated several commonly used and commercially available diversion valves or diversion boxes (Figure 4.7).

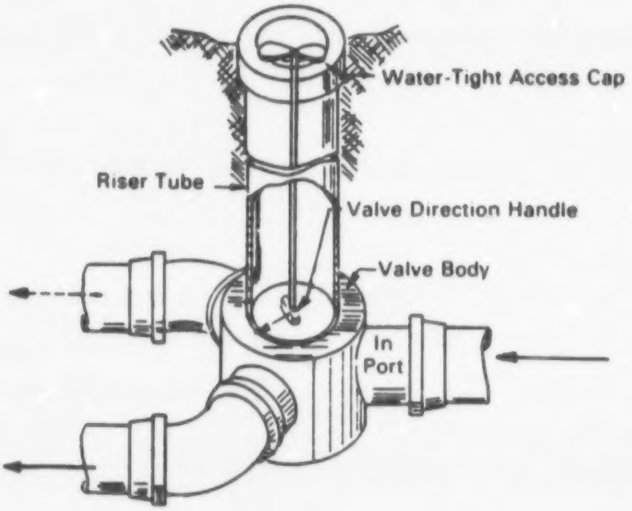
One diversion box uses a treated wood gate to divert the flow to the desired outlet pipe. The second uses 90° ells that can be moved from the horizontal to the vertical position to shut off flow. Caps or plugs have been used in place of elbows. The authors suggested that diversion boxes be well insulated for winter operation.

When using pressure distribution networks, the same authors reported that multiple pumps serving multiple fields are sometimes used.

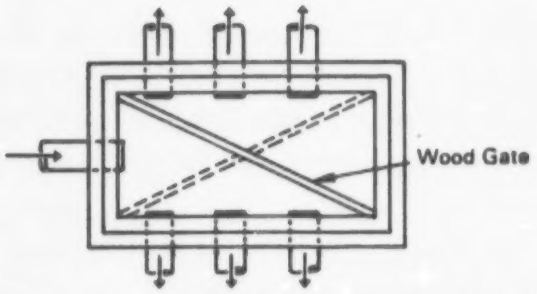
4.3 DRAINAGE FIELD ALTERNATIVES

4.3.1 Development of Drainage Field Systems

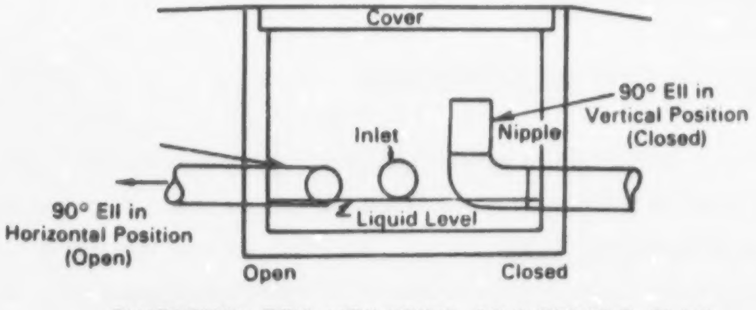
The earliest form of subsurface disposal consisted of cesspools. Otis et al. (1977) reported that because of the health hazards and nuisances posed by these rudimentary soil absorption systems, they began to be replaced during the early 1900's with subsurface irrigation systems - the forerunners of today's soil absorption field. They


Water-Tight Access Cap
Riser Tube
Valve Direction Handle
Valve Body
In Port

TYPICAL DIVERSION VALVE



DIVERSION BOX UTILIZING A TREATED WOOD GATE


Cover
90° Ell in
Vertical Position
(Closed)
Nipple
Inlet
90° Ell in
Horizontal Position
(Open)
Liquid Level
Open
Closed

DIVERSION BOX UTILIZING ADJUSTABLE ELLS

Figure 4.7

FLOW DIVERSION METHODS

apparently consisted of trenches dug wide enough to accommodate 85 to 150 mm diameter drain tile which was laid directly on the exposed trench bottom in open joint fashion. The joints were covered with tar paper before backfilling. No aggregate was used. A drain tile length of 12 m per person was considered sufficient despite the different soil conditions encountered. Some health departments suggested that in "dense" soils, the trench be excavated somewhat deeper and wider and the bottom filled with coarse aggregate before laying the tile. According to the authors, the purpose of the aggregate was to provide a porous media through which the septic tank effluent could flow. This increased the infiltration area and provided storage of the liquid until it could seep away.

The common occurrence of failures characterized by surfacing effluent led to the development of more refined design recommendations and procedures which finally culminated in the preparation of the Manual of Septic Tank Practice first published by the U.S. Public Health Service in 1957. The selection and design of drainage field systems today includes provision for the following:

1) absorption of all effluents generated;
2) a high level of treatment before the effluent reaches the groundwater; and
3) a long and useful drainage field life.

Several different designs of subsurface drainage field systems are being used successfully today. They include trenches and beds, mounds, and artificially drained systems. All are covered excavations filled with porous media and with one or more of the forms of effluent distribution systems discussed earlier. The porous media maintains exposure of the soil's infiltrative surface and provides storage for the wastewater until it can seep away into the surrounding soil. The capabilities of each drainage field design and the criteria employed vary with each method. They are each discussed in detail below.

4.3.2 Design of Trenches and Beds

Trench and bed systems are the most common disposal fields in use today. The US EPA (1980) has defined trenches as narrow excavations, usually 30 to 90 cm in width and up to 30 m or more in length. Only one distribution pipe is used per trench. The primary infiltration surfaces are the bottom and sidewalls of the excavation. The distribution pipe is laid on a bed of washed crushed rock or gravel which is 15 cm or more in depth. Additional rock is placed over the pipe and soil backfill is used to complete the trench. Beds are usually greater than 90 cm in width. More than one distribution pipe is used per bed. The primary infiltrative surface is the bottom of the excavation.

The sizing of trenches and beds is a function of the characteristics of the infiltrative surface. Several authors (Otis et al., 1977; Cotteral and Norris, 1969) have explained in detail how a clogging mat forms at the infiltrative surface, thereby reducing the rate of wastewater infiltration below the percolative capacity of the surrounding soil. They have emphasized why loading must be sized on the basis of the expected infiltrative capacity of the clogging mat. Otis (1982b) explained that the rate of infiltration through this mat is dependent upon:

1) the resistance of the clogging mat;
2) the hydrostatic pressure of the water ponded above; and
3) the capillarity of the underlying soil.

Measurement of equilibrium infiltration rates through clogged soil has been reported by Bouma (1975), Healy and Laak (1973) and US PHS (1974). Rates vary from soil to soil, but are often correlated with texture as shown in Table 4.2. These rates may, however, be affected significantly by soil structure, mineralogy, bulk density and various other factors (Otis, 1982). The values in Table 4.2 are an example of the type of relationship used to translate soil textural information into soil percolation rates and trench and bed loading rates.

Since both the horizontal and vertical surfaces of trenches and beds act as infiltrative surfaces, the sizing of disposal fields must consider both. Furthermore, as Otis (1982b) explained, since the hydraulic gradients and resistances of the clogging mats on the bottom and sidewalls are not likely to be the same, infiltration rates may be different. He suggested that the objective in design is to maximize the area of the surface expected to have the highest infiltration rate while assuring adequate treatment of wastewater and protection of the groundwater. The same author explained why this is difficult to predict:

1) clogging at the sidewall is likely to be less severe than at the bottom because suspended solids settle to the bottom while the rising and falling liquid levels cause dosing and resting cycles for the sidewalls; and

2) this may be offset by the lower hydraulic gradient across the sidewall because the gravity potential is zero and the hydrostatic pressure is less significant than at the bottom.

TABLE 4.2
RECOMMENDED SEPTIC TANK EFFLUENT LOADING RATES FOR TRENCHES AND BEDS
(US EPA, 1980)

Soil Texture	Percolation Rate (min/cm)	Bottom Area Application Rate[1] (cm/day)
Gravel, coarse sand	0.4	Not Suitable
Coarse to medium sand	0.4-2	5
Fine sand, loamy sand	2-6	3
Sandy loam, loam	6-12	2.5
Loam, porous silt loam	12-14	2
Silty clay loam, clay loam[2]	24-48	1[3]

1) May be suitable estimates for sidewall infiltration rates.
2) Soils without significant amounts of expandable clays.
3) Soils easily damaged during construction.

Because of the difficulty in predicting the highest infiltration rate, the author suggested the following design principle: the loading rate used for the bottom area should be the maximum rate applied to the sidewalls. Furthermore, he suggested that in humid regions where percolating rain water reduces the infiltration potential along the sidewall, the bottom area should be the principal infiltrative surface. He stated that in dry regions, the sidewall may be used to a greater extent because the sidewall infiltration potential remains high. The US EPA (1980) reported that common practice is not to include the first 15 cm of sidewall area as measured from the trench bottom; any exposed sidewall above 15 cm is suitable as infiltrative surface area for design purposes.

Additional criteria suggested in the same design manual included the following:

1) that trenches and beds should not be used for highly permeable soils with percolation rates greater than 0.4 min/cm unless a liner of less permeable loamy sand or sand textured soil is used;

2) that trenches and beds should be avoided where percolation rates are slower than 24 min/cm unless great care is taken during construction to resist compaction and smearing; and

3) that trenches and beds should not be used under any circumstances once the percolation rate is slower than 47 min/cm.

Machmeier (1979) and McGauhey and Winneberger (1965) suggested the use of a 10 to 15 cm sand layers in the bottom of the trench to protect the soil from compaction during placement of the aggregate and to expose infiltrative surface that would otherwise be covered by aggregate.

The US EPA (1980) suggested that the length of trenches and beds should depend on site characteristics but that the maximum length of distribution laterals is commonly restricted to 30 m. This latter design criterion is based on the concerns of root penetration, uneven settling or pipe breakage. Any of these could disrupt the flow down the pipe to render the remaining downstream length useless. Laak (1973), McGauhey and Winneberger (1965) and Otis et al. (1978) suggested that these concerns were unwarranted because the aggregate serves to transmit the wastewater as long as the trench or bed is level throughout its length.

The US EPA (1980) also suggested that the depth of trenches and beds should be determined by the location of the most permeable soil horizon and the presence of a flow restricting layer or the high water table elevation. The authors suggested that unless a deep, more permeable horizon exists, the trench or bed bottom elevation should be maintained at about 46 to 61 cm below the natural ground surface. Otis (1982b) suggested that these shallow trenches or beds have the following advantages over deep designs:

1) the upper soil horizons are more permeable than deeper ones because of greater plant and soil fauna activity and less clay due to eluviation;
2) evapotranspiration is greater;
3) upper horizons dry more quickly, thereby reducing construction delays; and
4) costs may be lower because of reduced excavation.

Machmeier (1979) reported that if beds were kept in continuous operation, freezing was not a problem even when frost penetration in the surrounding soil was much lower than the bottom of the trench or bed (1.5 m or more). In order to protect the system during cold weather operation, he suggested that the distribution pipe be gravel packed and the header pipes insulated where it is necessary for them to pass under driveways or other areas usually cleared of snow.

The US EPA (1980) reported that spacing between sidewalls could be as little as 46 cm. The suggested a spacing of 1.8 m, however, to facilitate construction and to provide a reserve area between trenches. Typical dimensions for trenches and beds have been summarized in Table 4.3.

TABLE 4.3

TYPICAL DIMENSIONS FOR TRENCHES AND BEDS (US EPA, 1980)

System	Width (cm)	Length[b] (m)	Bottom Depth[c] (cm)	Cover Thickness (cm)	Spacing[d] (m)
Trenches	30-100[a]	30	45-60	15 (min)	2
Beds	100	30	45-60	15 (min)	2

[a] Excavations generally should not be less than 30 cm wide because the sidewall may slough and infiltrate the aggregate.

[b] Length of lateral from distribution inlet manifold. May be greater if site characteristics demand.

[c] May be deeper if a more suitable horizon exists at greater depth and sufficient depth can be maintained between the bottom and seasonably high water table.

[d] From sidewall to sidewall. Trench spacing may be decreased because of soil flow net patterns, specifically for shallow trenches in sandy soils.

The US EPA (1980) stated that the aggregate placed below and around the distribution pipe has four functions:

1) to support the pipe and provide a media through which the wastewater can flow to the infiltrative surfaces;
2) to provide storage of peak flows;
3) to dissipate the energy of incoming wastewater; and
4) to support the sidewalls of the excavation and prevent its collapse.

They reported that the depth of the aggregate may vary; a minimum of 15 cm below the distribution pipe invert and 5 cm above the crown were suggested. They advised that greater depths may be used when it is desirable to increase the sidewall area and to increase the hydraulic head on the infiltrative surface.

The same design manual reported that gravel or crushed rock, 1.8 to 6.4 cm in diameter is usually used as the aggregate porous media. The authors suggested the following specifications for this media:

1) it should be durable and resistant to slaking and dissolution (a hardness of 3 or greater on the Moh's Scale of Hardness is suggested);

2) unless it is dolomitic, crushed limestone is unsuitable; and

3) the media should be washed to remove fines which might clog the infiltrative surface.

They suggested the use of untreated building paper, synthetic drainage fabric, marsh hay or straw to cover the top of the porous media prior to backfilling. Its purpose is to prevent backfilled soil from entering the media and filling the voids.

Although trenches and beds have been discussed interchangeably until now, there are several factors which govern the selection of one over the other. Otis (1982b) suggested that trenches are usually more desirable than beds for the following reasons:

1) trenches can provide up to five times more sidewall area than beds for identical bottom areas;

2) less soil damage occurs during construction because the excavation equipment can straddle the trenches, eliminating the need to drive on the infiltrative surface; and

3) trenches can follow the contours on sloping sites to maintain the infiltrative surface in the same soil horizon and to keep excavation to a minimum.

He further stated that beds may be acceptable in sands and loamy sands and where the site is relatively level. He suggested that beds should not be constructed on sites with slopes greater than 10% because the excavation becomes too deep on the upslope side.

4.3.3 Trench and Bed Design for Large Scale Application

The US EPA (1980) recommended that the design of trenches and beds for small communities and industrial applications generally should follow the same design principles as for single family dwellings with the following exceptions:

1) In community systems, peak flow estimates can be reduced because of flow attenuation, but contributions from infiltration through gravity collection systems must be included.

2) For industrial application the character of the wastewater becomes an important concern as previously discussed in Section 2.

There is very little published information on community and industrial septic tank disposal field systems. Diodato (1980) reported the development of such a system in a community of 200 housing units in Pennsylvania. The author summarizes the technical data used to design the beds. A gravity collection system using PVC pipe was installed. Cleanouts were placed at regular intervals thereby eliminating manholes. A pressure distribution system was selected to dose the effluent to the beds. This selection was based on the perception that pressure distribution establishes an aerobic/anaerobic equilibrium within the slime layer, thereby increasing the renovation capabilities and life span of the disposal systems.

Rubin and Carlile (1981) reviewed some of the problems associated with the development of subsurface disposal systems for rural communities and subdivisions. They discussed the failure of disposal fields used for wastewater disposal from fast-food restaurants in North Carolina. Reasons for most failures were two fold:

1) typically recommended wastewater flow design criteria (150 L/d per seat) proved to be grossly inadequate; and

2) high oil and grease concentrations (as high as 1720 mg/L) which clogged the disposal beds.

The authors suggested that because of the large area of these systems, a somewhat more detailed site investigation is necessary for large-scale septic tank disposal field systems. They emphasized that where large land areas are involved, there is often tremendous variation within the soils in those areas. They concluded their discussion with several case studies including the following:

1) the installation of a septic tank - disposal field system at a 200 employee garment factory effluent was fed to the beds with a pressure distribution system. Land area shortages were overcome with the use of water-conserving plumbing within the building; and

2) the installation of a septic tank disposal field system in a 400-student rural school; site constraints included poor soil and a very shallow water table (0.3 m) during some portions of the year.

Flexibility in operation is important for small community systems since disposal field failures usually have drastic consequences. Otis (1978) has suggested the use of a three-field system in which each disposal field contains 50% of the required absorption area. Two beds are able to provide the required infiltrative surface while the third is alternated in service on a semi-annual basis. Each field is therefore operated for one or two years and rested for six months to one year to rejuvenate. The third acts as a standby in case one fails.

4.3.4 Operating Characteristics of Trenches and Beds

Most design criteria for disposal field systems have been developed or modified through field observation of actual systems. Bouma (1975) measured soil moisture potentials and hydraulic conductivities by installing tensiometers in soil adjacent to disposal fields. Thirteen disposal fields ranging in age from 0.5 to 13 years were monitored. Soil types included coarse sand, gravelly sandy loam, silt loam, silty clay loam and clay. As a result of these and related studies, he made a series of recommendations for design which are summarized in Table 4.4.

TABLE 4.4

RECOMMENDATIONS FOR DISPOSAL FIELD DESIGN ON THE BASIS OF SOIL TYPES (Bouma, 1975)

Soil Type	Summary of Recommendations
Sands	1) maximum loading rate of 5 cm/day applied to bottom area only 2) dosing four times per day 3) use pressure distribution systems to achieve equal distribution for adequate pathogen removal (prior to formation of clogging layer) 4) siz[illegible] field on bottom area only 5) tre[illegible] or beds allowable
Sandy loams, loams	1) maximum loading rate of 3 cm/day 2) dosing once daily 3) equal distribution is important to avoid clogging 4) size the field on bottom area only 5) trenches are preferable to beds
Silt loams, some silty clay loams	1) maximum loading rate of 5 cm/day 2) dosing once daily 3) equal distribution desirable but not critical 4) size the field on bottom area only 5) only shallow trenches (45 to 60 cm) are acceptable; bed should not be used 6) construction of trenches only when soil is sufficiently dry that plastic conditions are not occurring
Clays, some silty clay loams	1) maximum loading rate of 1 cm/day 2) regular once daily dosing desirable but not esential 3) equal distribution desirable but not critical 4) size the field on bottom area only 5) only shallow trenches are acceptable; bed should not be used 6) construction of trenches only when soil is sufficiently dry that plastic conditions are not occurring

4.3.5 Design of Mound Systems

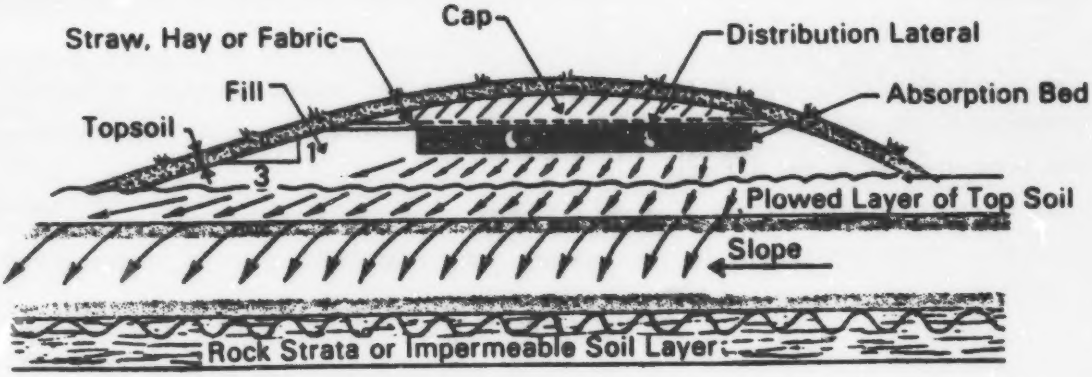
A mound system (illustrated in Figure 4.7) is a soil absorption system that is elevated above the natural soil surface in a suitable fill material. It was originally developed in North Dakota in the late 1940's where it became known as the NODAK disposal system (Witz et al., 1974). It was designed to overcome problems with slowly permeable soils and high water tables. The bed was constructed in coarse gravel placed over the original soil after the topsoil was removed. Initial problems of groundwater contamination and seepage, particularly during wet periods of the year, have been overcome through design modifications (US EPA, 1978).

The purpose of mound systems in use today is to overcome site retrictions that prohibit the use of trenches and beds (US EPA, 1978; Converse et al., 1978). These restrictions include:

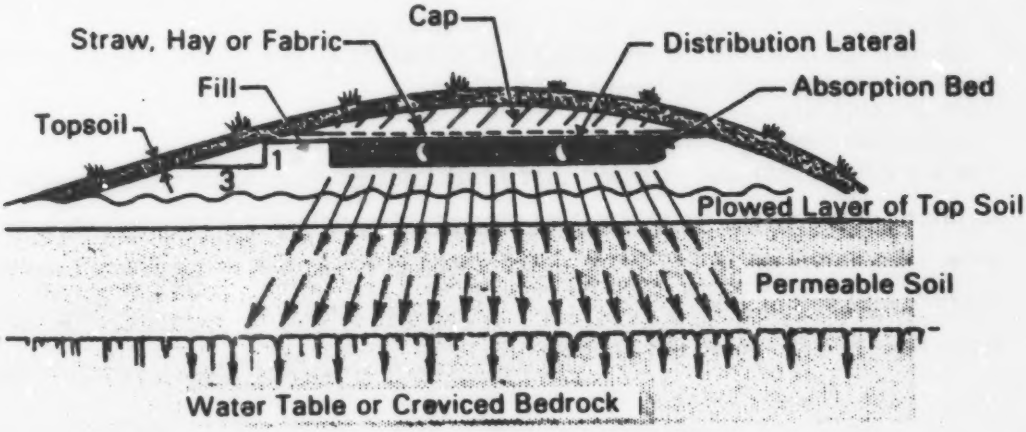
1) slowly permeable soils;
2) shallow permeable soils over creviced or porous bedrock; and
3) permeable soils with high water tables.

The US EPA (1980) has reported that in slowly permeable soils, the mound serves to improve absorption of the effluent by utilizing the more permeable topsoil and eliminating construction in the wetter and more slowly permeable subsoil, where smearing and compaction are often unavoidable. In permeable soils with insufficient depth to groundwater or creviced or porous bedrock, the fill material in the mound provides the necessary treatment of the wastewater.

As illustrated in Figure 4.8, the mound system consists of a suitable fill material, an absorption area, a distribution network, a cap, and topsoil. The effluent is pumped or siphoned into the absorption area through a distribution network located in the upper part of the coarse aggregate. It passes through the aggregate and infiltrates the fill material. Treatment of the wastewater occurs as it passes through the fill material and the unsaturated zone of the natural soil. The cap, usually a finer textured material than the fill, provides frost protection, sheds precipitation, and retains


Cap
Straw, Hay or Fabric
Distribution Lateral
Fill
Absorption Bed
Topsoil
1
3
Plowed Layer of Top Soil
Slope
Rock Strata or Impermeable Soil Layer

(a) Cross Section of a Mound System for Slowly Permeable Soil on a Sloping Site.


Cap
Straw, Hay or Fabric
Distribution Lateral
Fill
Absorption Bed
Topsoil
1
3
Plowed Layer of Top Soil
Permeable Soil
Water Table or Creviced Bedrock

(b) Cross Section of a Mound System for a Permeable Soil, with High Groundwater or Shallow Creviced Bedrock

Figure 4.8

TYPICAL MOUND SYSTEMS

moisture for a good vegetative cover. The topsoil provides a growth medium for the vegetation.

The US EPA (1980) summarized the site criteria for mound systems in current practice. These are presented in Table 4.5. The following site considerations were also recommended by the authors:

1) Slope limitations for mounds are more restrictive than for conventional systems, particularly for mounds on sites having slowly permeable soils;

2) The fill material and natural soil interface may represent an abrupt textural change that restricts downward percolation, increasing the changes for surface seepage; and

3) The acceptable depth to an impermeable layer or rock strata must be sufficient to channel the percolating wastewater away from the mound; it is governed by soil permeability, climate, slope and mound layout.

They noted that insufficient research information is available to give specific bed depths for the various site conditions.

The US EPA (1980) also stressed that mound design must begin with the selection of suitable fill material. Its infiltrative capacity determines the required absorption bed area. Converse et al. (1977) reported the successful use of medium texture sands, sandy loams, soil mixtures, bottom ash, strip mine spoil and slags. Loading characteristics have been summarized in Table 4.6.

TABLE 4.5
SITE CRITERIA FOR MOUND SYSTEMS (US EPA, 1980)

Item	Criteria
Landscape Position	Well drained areas, level or sloping. Crests of slopes or convex slopes most desirable. Avoid depressions, bases of slopes and concave slopes unless suitable drainage is provided.
Slope	0 to 6% for soils with percolation rates slower than 23 min/cm 0 to 12% for soils with percolation rates faster than 23 min/cm
Typical Horizontal Separation Distances from Edge of Basal Area	
Water Supply Wells	15 to 30 m
Surface Waters, Springs	15 to 30 m
Escarpments	3 to 6 m
Boundary of Property	1.5 to 3 m
Building Foundations	3 to 6 m (9 m when located upslope from a building in slowly permeable soils.)
Soil	
Profile Description	Soils with a well developed and relatively undisturbed A horizon (topsoil) are preferable. Old filled areas should be carefully investigated for abrupt textural changes that would affect water movement. Newly filled areas should be avoided until proper settlement occurs.
Unsaturated Depth	50 to 60 cm of unsaturated soil should exist between the original soil surface and seasonally saturated horizons or pervious or creviced bedrock.
Depth to Impermeable Barrier	1 to 1.5 m[b]
Percolation Rate	0 to 47 min/cm measured at 5 to 8 cm[c]

---

[a] These are present limits used in Wisconsin established to coincide with slope classes used by the Soil Conversation Service in soil mapping. Mounds have been sited on slopes greater than these, but experience is limited.

[b] Acceptable depth is site dependent.

[c] Tests are run at 50 cm unless water table is 50 cm, in which case test is run at 40 cm. In shallow soils over pervious or creviced bedrock, tests are run at 30 cm.

TABLE 4.6

COMMONLY USED FILL MATERIALS AND THEIR DESIGN INFILTRATION RATES (Converse et al., 1977)

Fill Material	Particle Size Characteristics		Design Infiltration Rate (cm/day)
Medium Sand	25% 30-35% 5-10%	0.25-2.0 mm 0.05-0.25 mm 0.002-0.05 mm	5.0
Sandy Loam	5-15%	Clay Content	2.5
Sand/Sandy Loam Mixture	88-93% 7-12%	Sand Finer Grained Material	5.0
Bottom Ash		-	5.0

The absorption area within the mound system can be either a bed or a series of trenches (US EPA, 1980). The authors reported that beds are typically used in smaller systems because they are easier to construct and that the shape of the bed depends on the permeability of the natural soil and the slope of the site. A rectangular bed with the long axis parallel to the slope contour is usually preferred to minimize the risk of seepage from the base of the mound. The authors suggested that if the natural soil percolation rate is slower than 24 min/cm, the bed should be made narrow and extended along the contour as far as possible. In soils with percolation rates faster than 24 min/cm, the bed can be square if the water table is greater than 90 cm below the ground surface.

The same authors summarized additional design criteria for mound systems:

1) The dimensions of the mound are dependent on the size and shape of the absorption bed, the permeability of the natural soil, slope of the site and depth of fill below the bed; typical bed dimensions are summarized in Figure 4.9 and Table 4.7;

2) The downslope setback (Figure 4.8) is dependent on the permeability of the natural soil (refer to Table 4.8); and

3) The basal area must be sufficiently large to absorb the wastewater before it reaches the perimeter of the mound; otherwise surface seepage will result.

TABLE 4.7

DIMENSIONS FOR MOUND SYSTEMS[a]

(Converse, 1978)

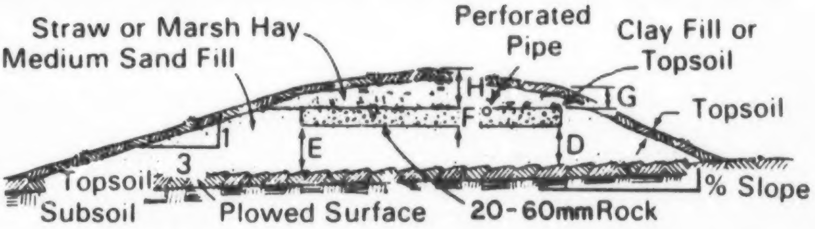
Item	Dimension
Mound Height	
Fill Depth (D), cm	30 (min)[b]
Absorption Bed Depth (F), cm	23 (min)
Cap at Edge of Bed (G), cm	30[c]
Cap at Centre of Bed (H), cm	45[c]
Mound Perimeter	
Downslope Setback (I)	Depends on Soil Permeability
Upslope Setback (J), cm	300[d]
Side Slope Setback (K), cm	300[d]
Side Slopes	No Steeper than 3:1

[a] Letters refer to lettered dimensions in Figure 4.8.

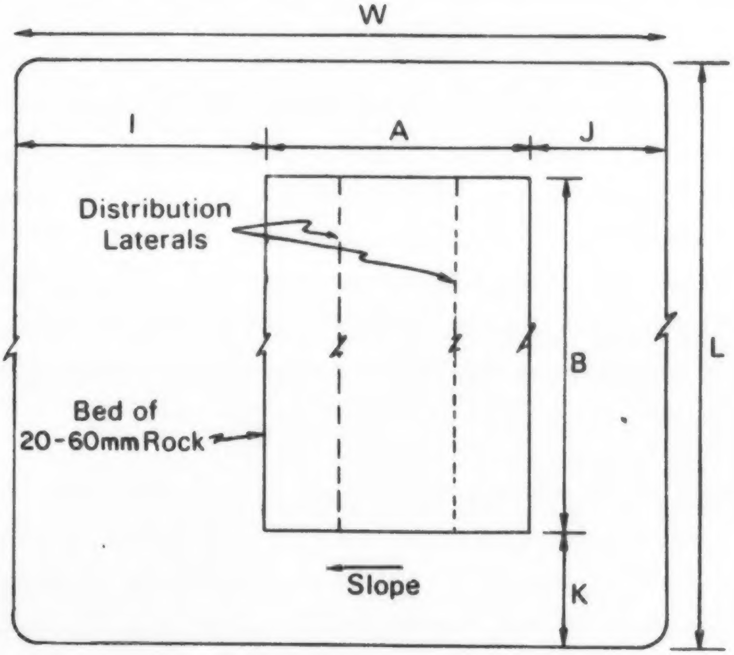
[b] On sloping sites, this depth will increase downslope to maintain a level bed. In shallow soils where groundwater contamination is a concern, the fill depth should be increased to 60 cm.

[c] A 10-15 cm depth of quality topsoil is included. This depth can be decreased by 15 cm in areas with mild winters. If depths less than 30 cm are used, erosion after construction must be avoided so sufficient soil covers the porous media.

[d] Based on 3:1 side slopes. On sloping sites, (J) will be less if the 3:1 side slope is maintained.


Straw or Marsh Hay
Medium Sand Fill
Perforated Pipe
Clay Fill or Topsoil
H
G
F
Topsoil
1
E
D
3
Topsoil
% Slope
Subsoil
Plowed Surface
20-60mm Rock

(A) Cross Section


W
I
A
J
Distribution Laterals
B
L
Bed of 20-60mm Rock
Slope
K

(B) Plan View

Figure 4.9

MOUND DIMENSIONS (USEPA, 1980)

TABLE 4.8
INFILTRATION RATES FOR DETERMINING MOUND BASAL AREA
(US EPA, 1978)

Natural Soil Texture	Percolation Rate min/25 mm	Infiltration Rate cm/day
Sand, Sandy Loam	0-30	5.0
Loams, Silt Loams	30-45	3.0
Silt Loams, Silty Clay Loams	45-60	2.0
Clay Loams, Clay	60-120	1.0

Converse et al. (1977) reported that pressure distribution systems are favoured in mound systems because the effluent is spread more uniformly over the entire absorption area thus eliminating saturated flow and short circuiting. Converse (1978) recommended frequent dosing (four times daily). The authors of the US EPA (1980) Design Manual stressed the importance of careful construction techniques in the proper functioning of mound systems. The manual presented a recommended stepwise procedure for construction.

4.3.6 Mound Design for Large Scale Application

As with trenches and beds, the design of mound systems for larger flows follows the same design practices as for single family unit disposal fields with the exceptions noted earlier. The US EPA (1980) recommended the use of a three bed system to provide flexibility in operation.

The same authors suggested that where mounds are used on sloping sites or in slowly permeable soils, the absorption area be broken up into a series of trenches or smaller beds. They listed the following advantages:

1) on sloping sites, beds can be tiered to reduce the quantity of fill required; and
2) in slowly permeable soils the effluent is distributed over a much wider area.

They recommended that depths of fill material below beds on slopes should not exceed 1.3 to 1.7 m because differential settling will cause uneven bed settling. They suggested that spacing of beds or trenches should be sufficient so that the wastewater contributed from one trench or bed is absorbed by the natural soil before it reaches the lower trench or bed.

4.3.7 Operating Characteristics of Mounds

Harkin et al. (1979) evaluated thirty-three mound systems in Wisconsin installed by commercial contractors. They reported the following results:

1) Although all were properly designed, sized and installed, the rate of application of effluent was much higher than recommended; despite high loading rates there was no evidence of serious bacterial or chemical contamination of groundwater even in permeable subsoil systems;

2) No systems examined had a clogging layer;

3) Three systems failed because of freezing; the authors stressed the importance of an adequate cap or cover to provide thermal insulation;

4) Denitrification removed about 44 percent of the nitrate formed; better removal of fixed nitrogen was observed in systems receiving frequent small doses than in systems receiving occasional large doses; and

5) No evidence of phosphorus enrichment.

Peterson and Fritton (1979) estimated from the results of a questionnaire survey that six to seven percent of mound systems in the State of Pennsylvania were malfunctioning. Causes included unsuitable soil characteristics, improper site selection, construction problems and lack of adequate maintenance. They stressed the importance of extreme care in the construction and maintenance of mound systems.

Simmons and Magdoff (1979a) reported that temperatures in mounds located in Vermont fell below freezing during a few months in winter.

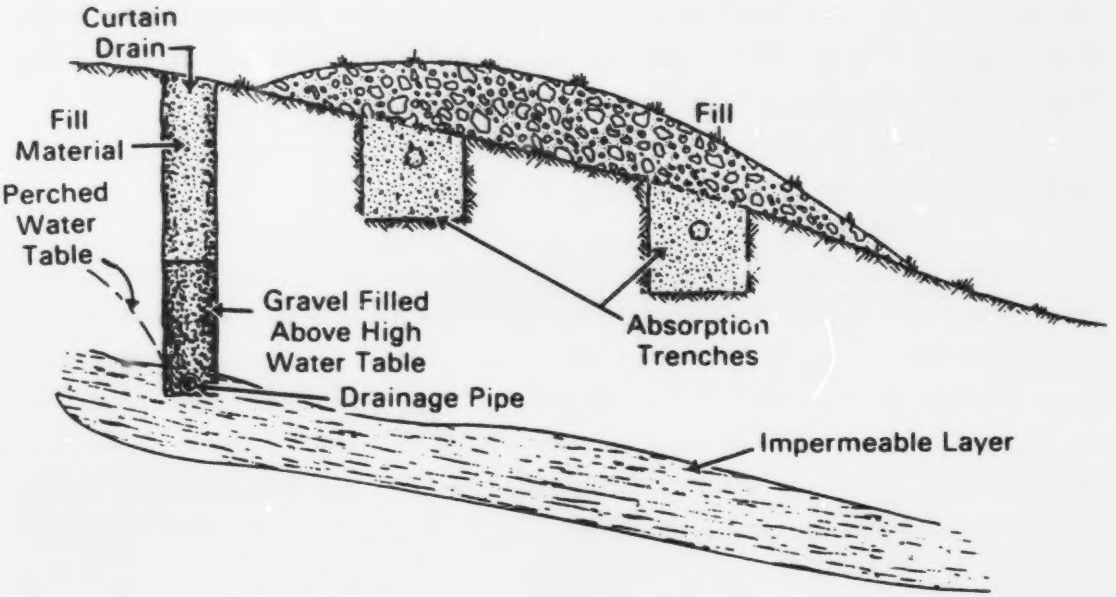
4.3.8 Artificially Drained Systems

The US EPA (1980) reported that where high water tables limit the use of trenches or beds, vertical drains, curtain drains and underdrains are commonly used. The authors reported that curtain and vertical drains are used to lower perched water tables and are most effective where the perched water is moving laterally under the soil absorption site.

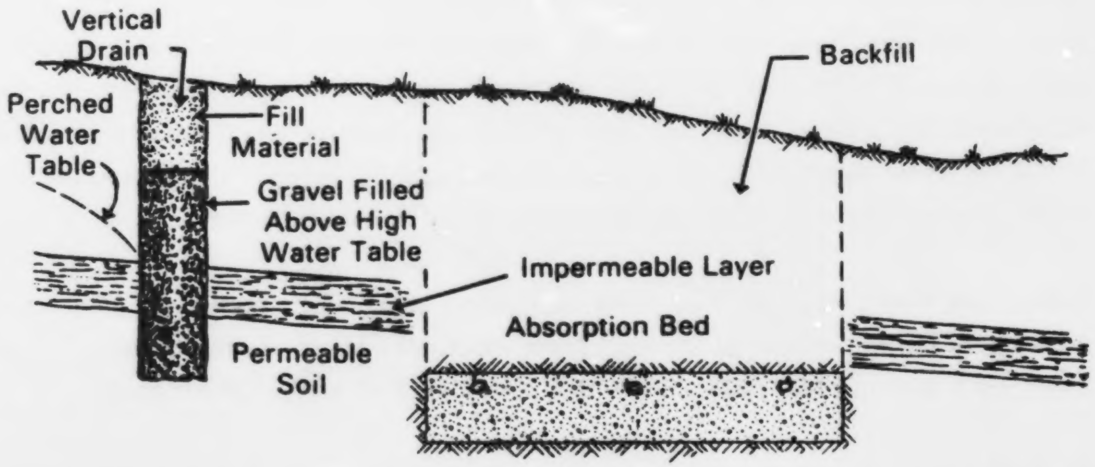
Curtain drains were defined by the authors as trench excavations upstream of the absorption area. Into these trenches is placed perforated drainage pipe to intercept the groundwater moving into the area. The drains are either pumped out (on level sites) or brought to the surface downslope to remove collected water. A typical cross-section is illustrated in Figure 4.10.

If the restrictive layer creating the high water table is thin and overlies permeable soil, the same authors suggested that vertical drains may be used. They were defined as trench excavations made through the restrictive layer into the more permeable soil below and then backfilled with porous material (Figure 4.9). Groundwater moving towards the excavation is able to drain into the underlying soil.

A third alternative described in the design manual was the use of vertical drains - similar to curtain drains in construction, and used where shallow water tables exist at a depth of 1.2 to 1.5 m. The authors reported that depth and spacing of the drains are determined by the soil and water table characteristics.


Curtain Drain
Fill Material
Perched Water Table
Gravel Filled Above High Water Table
Drainage Pipe
Fill
Absorption Trenches
Impermeable Layer

CURTAIN DRAIN


Vertical Drain
Perched Water Table
Fill Material
Gravel Filled Above High Water Table
Backfill
Impermeable Layer
Absorption Bed
Permeable Soil

VERTICAL DRAIN

Figure 4.10

ARTIFICIALLY DRAINED SYSTEMS (USEPA, 1980)

The design of artificially drained systems is discussed in detail in the US EPA (1980) Design Manual. The authors stressed the importance of characterizing the high water table problem adequately prior to implementing a drainage alternative.

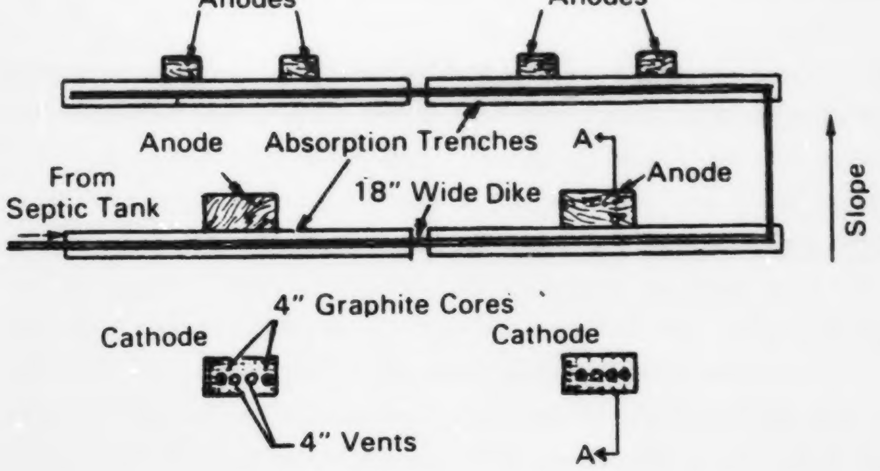
4.3.9 Other Disposal Field Alternatives

The fill system, described in detail in US EPA (1980), is a modification of conventional trenches and beds. It was reported as being useful on sites with slowly permeable soils overlying sands and sandy loams and where construction of a conventional system below the tight soil horizons may be impractical. The impermeable layer of surface soil is physically removed and replaced with a sandy fill material and a conventional trench or bed constructed within the fill. Site elevation and design criteria for fill systems are discussed in the same design manual.

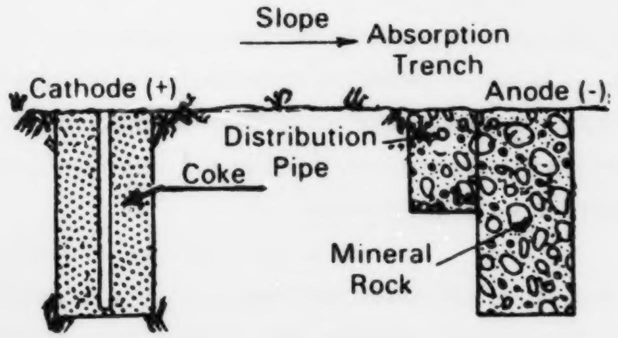
Several authors (US EPA, 1980; Casagrande, 1952; Coolbroth, 1976) have reported the use of the electro-osmosis process in very impervious soils. Coolbroth (1976) described the principle of electro osmosis. Groundwater between fine silt and clay soil grains is made up of three layers, including an uncharged inner core, a negatively charged surface layer fixed to the soil grains, and a positively charged intermediate layer between the fixed layer and inner core. Normally the positive and negative water layers adhere to each other, but when an electric potential is applied between nearby electrodes, the positive layer will flow towards the negative electrode dragging the uncharged water core with it. As a result, the increased movement of the underground water is far greater than that imposed by the natural hydraulic gradient.

Casagrande (1952) reported that steel well points serve as cathodes and steel rods driven between wells serve as anodes. These electrodes are commonly installed 4.5 m apart and a current of 20 to 30 amps with a potential of 30 to 180 volts is applied.

Peterson (1972) recently patented a similar technique for use in soils with percolation rates slower than 24 min/cm (Figure 4.11). A galvanic cell, constructed of natural materials and requiring no external power source is capable of generating a 0.7 to 1.3 volt potential. Conventional absorption trenches are constructed and a mineral rock-filled anode is installed immediately adjacent to the trench. Coke-filled cathodes with


Anodes
Anodes
Anode
Absorption Trenches
A
Anode
From
Septic Tank
18" Wide Dike
Slope
4" Graphite Cores
Cathode
Cathode
4" Vents
A

Plan View


Slope
Absorption
Trench
Cathode (+)
Anode (-)
Distribution
Pipe
Coke
Mineral
Rock

Section A-A

Figure 4.11

TYPICAL
ELECTOR-OSMOSIS SYSTEM (USEPA, 1980)

graphite cores are installed at some distance from the trench. The design and construction of electro-osmosis systems is done by licensees.

Beer et al. (1977) reported no beneficial effects of electro-osmosis systems over conventional trenches. Effert and Beer (1984) further substantiated these earlier conclusions.

4.4 SOIL PERMEABILITY EVALUATIONS

As explained earlier, the hydraulic conductivity of the clogging layer and the soil below it is one of the most important variables in the design of disposal fields. It is also the most difficult value to predict or to measure accurately. Several methods have been developed to provide approximations. The most commonly used test is the percolation test. The US EPA (1980) stated that when conducted properly, the test gives an approximate measure of the soil's saturated hydraulic conductivity. The percolation of wastewater through the clogging layer and into the soil below usually occurs in unsaturated conditions. Therefore, empirical factors must be applied to percolation test values to characterize unsaturated hydraulic conditions. The same authors reported that these empirical values can vary dramatically with changes in soil characteristics and moisture content.

Although several percolation test methods are used in practice, the most common is the falling head test. It is also referred to as the auger hole percolation test. It is carried out by augering or digging a hole 15 to 30 cm in diameter to the depth of the proposed disposal field, pouring water into the hole to a depth greater than 30 cm and measuring the rate that the water level drops.

The percolation test has often been criticized because of its variability and failure to measure hydraulic conductivity accurately. Percolation tests done at the same site may vary by 90 percent or more (Bouma, 1971; Healy and Laak, 1973). In laboratory and field studies to determine the effect of soil properties and test methods on the percolation rate, Healy and Laak (1973) discovered that in fine-grained soil, the percolation rate was dependent on the size and shape of the hole, the permeability, and the ambient capillarity of the soil at the time the test was run. Long-term soaking did not eliminate the effect of capillarity.

The field experience of Healy and Laak (1973) also showed that the percolation rate was not sensitive to soil permeability but was primarily a function of the capillary potential of the soil at the time of the test. Capillary potential seemed to be more dependent on seasonal rainfall conditions than on the position of the water table. The authors concluded that the percolation rate was not a measure of the effective absorption properties of a soil and should not, therefore, be used as a basis for calculating hydraulic conductivities.

Hentzsche et al. (1982) suggested that the factors influencing the percolation values included hole diameter, water depth, initial soil moisture conditions, slaking of bottom and sidewalls, air entrapment, and the general care and attention of the technician taking the test. They stressed the need to refine soils evaluation techniques and developed a soil permeability evaluation procedure based primarily on soil texture. Closely associated with soil texture in importance, according to the same authors are bulk density and coarse fragment content.

Several researchers have discovered a relationship between soil physical characteristics and percolation rates. Derr et al. (1969) reported significant correlation coefficients for percolation rates with drainage class and percent coarse fragments, silt, and clay in the subsoil. Barbarick et al. (1976) discovered significant correlation between percolation rate and the percentage of silt, clay and sand in the subsoil. They suggested the following modifications to the percolation test procedures:

1) use hand tools to construct test holes;
2) employ four to six test holes at each location so measurements are replicated;
3) dig test holes at least 60 cm deep;
4) use a test-hole diameter of 30 cm;
5) add 5 cm of pea gravel to each test hole;
6) pre-soak each test hole for 6 to 20 hours for coarse-textured and fine-textured soils, respectively;

7) use a float gauge or similar device to measure percolation rate;
8) Establish the water level at 20 cm above the bottom of the test hole before measuring the percolation rate;
9) use a 2.5 cm rate of water fall as the percolation rate; and
10) consult detailed soil maps before selecting septic system locations.

The US EPA (1980) suggested that the percolation test can be useful if used together with soil borings data. The authors reported that the test could be used to rank the relative hydraulic conductivity and when used with Table 4.9 would yield representative results. They suggested the following procedure:

1) if percolation test results agree with the table, the test and soil boring data are probably correct and can be used in design; and

2) if the results do not agree, either the test was run improperly or soil structure or clay mineralogy have had a significant effect on the hydraulic conductivity and the necessary allowances should be made.

Several researchers have suggested alternatives to the percolation test which they deemed more accurate descriptions of hydraulic conductivity. Baker and Bouma (1977) described the crust test procedure. They used it to measure hydraulic conductivity (K) directly in silt loam soils and described the results. They suggested three alternative procedures for sizing the disposal fields:

1) The first is to direct on-site testing of permeability with the use of the crust test method to yield a K-curve for the site.

2) The second requires a knowledge of the mean hydraulic conductivity characteristics of soils at the series level and information on its variability; on-site determinations in specific soil series would be necessary and the data for that soil series would be used for evaluation.

3) The third alternative would eliminate any on-site field determination by the use of detailed soil maps; the reliability of this method is a function of the accuracy of soil series maps.

Vinson and Mahar (1981) described the use of subsurface neutron probes (devices used to detect fluid bearing zones for petroleum exploration) to estimate soil percolation rates. The results from ten laboratory tests using a column of well-graded sand showed very good correlation between calculated and measured percolation rates. The viability of the procedure for layered soil profiles or shallow groundwater tables was not evaluated.

TABLE 4.9

ESTIMATED HYDRAULIC CHARACTERISTICS OF SOIL

(Bouma, 1975)

Soil Texture	Permeability (cm/h)	Percolation (min/cm)
Sand	greater than 15.25	less than 4
Sandy loams Porous silt loams Silty clay loams	0.5 to 15.0	4 to 18
Clays, compact Silt loams Silty clay loams	less than 0.5	greater than 18

4.5 BED FAILURES

Once installed, soil disposal fields usually require very little or no attention so long as the wastewater being applied is nearly free of settleable solids, greases, fats and oils.

Occasionally, however, failure of disposal fields does occur. Causes of failure can be complex, resulting from poor siting, poor design, poor construction, poor maintenance, hydraulic overloading or various combinations of these (US EPA, 1980).

Several investigations have shown that the greatest chance of system failure occurs within the first three years of operation and that most of these failures are due to faulty construction practices (Scalf et al., 1977). Improper use of construction equipment can compact the absorption area and reduce the soil's liquid absorption capacity. The situation is further compounded during periods of excess soil moisture. Otis and Bouma (1973) reported the smearing of soil absorption faces by heavy rainfall or silt-laden wind while trenches were open during construction.

Reference was made earlier to the clogging of the infiltrative surface with large amounts of fats and greases or settleable solids. Systems serving commercial units, particularly restaurants, are subject to clogging unless grease traps are installed and maintained adequately. Lack of maintenance of septic tanks in the form of periodic sludge removal may result in sufficient settleable solids and scum carryover into the beds to seal the infiltrative surface. In all these cases some method of rejuvenation is required.

Simons and Magdoff (1979b) conducted laboratory experiments using sand columns loaded with septic tank effluent. They studied the effects of effluent quality, aeration status, temperature and loading rate on the clogging rates. Their experiments provided evidence that low temperatures (1 to 3°C) induce rapid infiltration rate declines. They concluded that a safe loading rate for a sand fill infiltrative surface at low temperatures may be 2.0 cm/day.

Daniel and Bouma (1974) studied clogging as a function of effluent quality in slowly permeable silt loam soils by using undisturbed 60 cm long cores in laboratory column experiments. They concluded that clogging of soil pores was closely related to the quality of suspended solids in the effluent. Aerobic effluent clogged the soil more severely than anaerobic effluent. The aerobic effluent contained suspended solids having a smaller particle size than those in the anaerobic effluent. They concluded that these finely divided particles were more easily able to penetrate the soil pore system to form "bottlenecks", thus reducing the overall permeability.

Occasionally failures are due to hydraulic overloading. This may be the result of excessive water use, plumbing leaks, or foundation drain discharges or it may be caused by poor drainage or seasonally high water table conditions.

A final cause of failure in disposal fields has been attributed to chemicals in the wastewater which may destroy or contribute to the destruction of soil structure. The structure of soil, especially in clay soils, contributes significantly to permeability. Anything that breaks down a soil's structure usually results in significant loss of permeability. Soil structure is generally sensitive to balances of divalent calcium and magnesium and monovalent sodium and potassium in infiltrating waters. Divalent cations usually enhance soil structure while monovalent ions compete with those processes, particularly in clayey soils. Sodium, in particular has received a great deal of attention because of its widespread use in water softeners.

The role played by sodium in the clogging of disposal fields is controversial. Winneberger and Weinberg (1976), noted that a search of literature disclosed that losses of permeability occurred when infiltrating fresh waters contained high concentrations of sodium but when the same high concentrations of sodium were in sewages, permeabilities of the soils were not much changed. The authors hypothesized that positively charged sodium ions were preferentially sorbed onto negatively charged suspended solids in septic sewages rather than onto negatively charged soil particles. The authors discovered through experiment that when a septic tank effluent with approximately 100 mg/L of suspended solids was fortified with over 1000 mg/L sodium, the added sodium was all adsorbed by the suspended solids. They also discovered that septic tank sludge had a significant capacity for adsorption of sodium. They used these results to support the hypothesis that sodium bicarbonate acts as a flocculant aid when added to the contents of a septic tank. The attachment of positively charged sodium ions to the negatively charged sewage particles neutralizes the particle charges, thereby enhancing gravity settling.

Just as the causes of bed failure are numerous and varied, so are the solutions. The US EPA (1980) has suggested a useful troubleshooting checklist to overcome the problem (see Figure 4.11). The authors have placed all bed failures into two major categories - either occasional failures or continuous failures. Occasional failures are exhibited by periodic seepage on the ground surface, sluggish drains or plumbing backups, usually during periods of heavy rainfall or snowmelt. Continuous failures often have the same symptoms but on a continuous basis. Some failures may have no surface effects but manifest themselves by causing groundwater contamination.

Occasional failures are usually simple to diagnose and easily rectified (US EPA, 1980). Because they are occasional in nature, bed sizing and construction practices can usually be eliminated as causes. Failure is the result of poor siting, poor maintenance or hydraulic overloading.

The authors of US EPA(1980) have suggested the following logical sequence of site investigations for occasional failures:

1) Investigate surface grading and landscape position to check for poor surface drainage conditions; corrective action includes regrading or filling of low areas.

2) Take soil borings to check for seasonally high groundwater tables; corrective actions may include installation of artificial drains.

3) Check for excessive water use, plumbing leaks or foundation drain discharges; corrective actions are noted in Figure 4.12.

4) Check for infiltration and groundwater leakage into the septic tank; Corrective actions include emptying of the septic tank and repair of leaks.

Continuous failures are more difficult to diagnose and correct. The authors (US EPA, 1980) suggested that learning the age of the system when failure occurs is quite useful in isolating the cause. If failure has occurred within the first year of operation, the cause is probably due to poor siting, design or construction. They have suggested

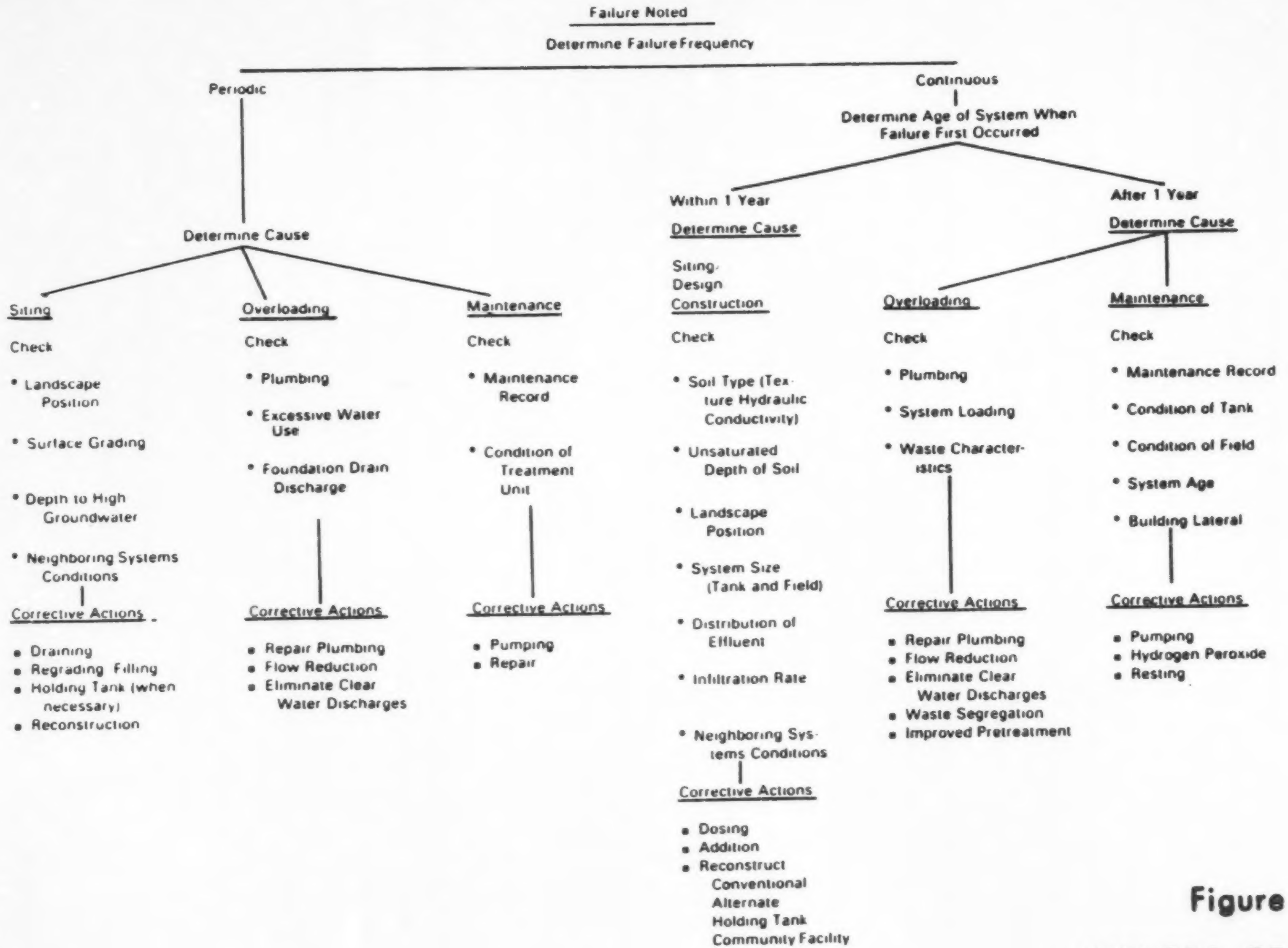

Failure Noted
Determine Failure Frequency
Periodic
Continuous
Determine Age of System When Failure First Occurred
Determine Cause
Siting
Check
• Landscape Position
• Surface Grading
• Depth to High Groundwater
• Neighboring Systems Conditions
Corrective Actions
• Draining
• Regrading Filling
• Holding Tank (when necessary)
• Reconstruction
Overloading
Check
• Plumbing
• Excessive Water Use
• Foundation Drain Discharge
Corrective Actions
• Repair Plumbing
• Flow Reduction
• Eliminate Clear Water Discharges
Maintenance
Check
• Maintenance Record
• Condition of Treatment Unit
Corrective Actions
• Pumping
• Repair
Within 1 Year
Determine Cause
Siting, Design Construction
Check
• Soil Type (Texture Hydraulic Conductivity)
• Unsaturated Depth of Soil
• Landscape Position
• System Size (Tank and Field)
• Distribution of Effluent
• Infiltration Rate
• Neighboring Systems Conditions
Corrective Actions
• Dosing
• Addition
• Reconstruct
Conventional
Alternate
Holding Tank
Community Facility
After 1 Year
Determine Cause
Overloading
Check
• Plumbing
• System Loading
• Waste Characteristics
Corrective Actions
• Repair Plumbing
• Flow Reduction
• Eliminate Clear Water Discharges
• Waste Segregation
• Improved Pretreatment
Maintenance
Check
• Maintenance Record
• Condition of Tank
• Condition of Field
• System Age
• Building Lateral
Corrective Actions
• Pumping
• Hydrogen Peroxide
• Resting

Figure 4.12

METHODS OF SOIL
ABSORPTION FIELD REHABILITATION (USEPA, 1980)

checking the performance of neighbouring systems installed in similar soils, determining their loading rates and bed sizes, and comparing them with the failed system. If bed sizing is inadequate, new infiltration area can be added to enlarge the disposal field. Poor distribution within the disposal field may be the problem. This can usually be corrected by better dosing of the beds. Finally, poor construction practices may have caused complete or partial sealing of the infiltrative surfaces during installation. Corrective action usually requires complete reconstruction or the addition of new beds.

The authors suggested that if the disposal field had many years of trouble-free service when failure occurred, the cause is usually hydraulic overloading or poor maintenance of the septic tank. Methods of checking and correcting hydraulic overloadings were discussed previously for occasional failures.

The clogging of beds by grease, fats and suspended solids from improperly maintained grease traps and septic tanks has been previously referenced. It is a frequent cause of permanent failure in older beds. In such cases, the infiltrative surface can sometimes be rejuvenated by oxidizing the clogging mat. This is normally accomplished by allowing the system to drain and rest for several months. This promotes the return of aerobic conditions conducive to the destruction of the clogging slime layer which forms at the soil interface. The use of dual or multi disposal field systems allows this rejuvenation on a periodic basis and is a major advantage of such systems.

The use of hydrogen peroxide, a strong chemical oxident, has been tested recently and a process utilizing that chemical has been licensed for application to failed disposal field rejuvenation. It is a strong oxidizing agent and a 30% solution will rapidly attack most organic matter including human tissue. Rejuvenation of beds with hydrogen peroxide is due to the following activities (Bishop and Logsdon, 1981):

1) It chemically oxidizes the black insoluble sulphides and some of the accumulated organic matter.

2) The vigorous effervescence produced by the chemical decomposition of peroxide to water and gaseous oxygen may agitate the soil and break up the clogging layer.

3) The released oxygen helps to aerate the soil.

Hill (1982) described the procedures for utilizing the POROX (hydrogen peroxide) process for rejuvenation of failed beds.

Bishop and Logsdon (1981) evaluated, through soil column experiments, the effectiveness of hydrogen peroxide. They concluded that although rejuvenation of sands and sandy loams was easy to achieve, it was more difficult in finer loam soils, probably because their finer texture requires greater agitation from effervescence and because the organic matter already present in loam soils places additional demand on the available hydrogen peroxide. Hargetl et al. (1984) reported that hydrogen perioxide can do sewer damage to the infiltrative capacity of non-sandy soils and they made a recommendation against its use.

As earlier mentioned, the role, if any, played by water softener salts in the failure of disposal fields is not fully known. The US EPA (1980) has suggested that avoidance is the best solution and has therefore recommended the disposal of backwash and rinse water from water softeners to a separate disposal field.

One cause of bed failures, particularly in Canada, is freezing of the liquid during the winter months. Cousin (1973) studied the reasons for failure of septic tank systems in Manitoba and the methods of preventing freezing during the winter months. He cited the following causes of bed failures:

1) failure to protect construction in freezing weather;
2) insufficient bed size, resulting in prolonged ponding and subsequent freezing;
3) short term abandonment of the disposal field in winter; and
4) vehicular traffic over the disposal field area.

He reported that the successful operation of a mound system during the winter months required the feeding of small doses to the bed. This served to reduce the amount of sewage requiring disposal at any one pumping cycle which shortened the time between cycles, thereby lessening the chance of frost penetration.

In a study carried out in Manitoba in 1949 and 1950, Ducharme (1950) evaluated a number of septic tank-disposal field systems near Winnipeg, Manitoba. With the use of thermocouples, he monitored the temperatures within the septic tanks and the disposal trenches during the winter months. He made the following conclusions and recommendations:

1) Properly designed and installed systems operated very successfully even during severe outside temperatures as low as -40°C; temperatures in the septic tank remained at 4°C or higher during the winter and temperatures in the disposal field were in the range of 0°C.

2) Where failure did occur due to bed freezing in the winter it was due either to the use of a trickle feed system or due to flooding and inundation of the beds and subsequent freezing.

The author recommended the use of syphon dosing systems and emphasized the need for proper sizing and installation of disposal fields. Lamphere (1952) reported very similar temperature results during winter operation of some experimented tile systems in Fairbanks, Alaska.

Osteraas (1983) reported that Norwegian experience indicated very few cases of bed freezing of disposal fields in continuous use. Where it did occur, accumulated sludge in pipes and joints usually started the frezing process. Bed underloading and irregular dosing were also reported causes. Laboratory results indicated that frozen absorption fields will work even when the temperature in the soil is below freezing. The author developed a procedure for estimating disposal field insulation requirements for winter operation.

SECTION 5

MONITORING SYSTEMS

5.1 SEPTIC TANK MONITORING

The importance of regular inspection of septic tanks to monitor scum thicknesses and sludge depths has been discussed previously. The US EPA (1980) has suggested that tanks should be inspected at intervals of no more than every two years. The authors further recommend that the inlet and outlet structures and key joints should be inspected for damage after each pump-out. They also described several practical methods of measuring scum and sludge depths in the septic tank.

5.2 MONITORING BED AND SOIL WATER LEVELS

Inspection pipes are sometimes installed within soil disposal systems (US EPA, 1980). They consist of open-ended pipes extending vertically from the bottom infiltrative surface of the system up to or above the final grade. They provide limited access to observe depth of ponding and performance of the disposal field system and they serve as a means for locating the subsurface field. When installed, the portion of the pipe within the gravel should be perforated to permit a free flow of water (US EPA, 1980). Since these inspection pipes can also act as cold air conductors, an insulation plug should be installed during the winter months.

The importance of monitoring soil water levels to determine the cause of occasional bed failures was discussed in the previous Section. This can be done very simply with the use of soil bore holes dug with a hand auger in an area adjacent to the beds. The use of soil bore holes may also be useful where artificial drains are installed to reduce seasonally high groundwater. They provide a permanent monitoring tool. In such cases, the bore holes can be made more permanent by installing perforated piezometer pipes.

Most literature reports of groundwater monitoring systems have concentrated on the evaluation of water quality. Viraraghavan and Warnock (1976) described a groundwater monitoring system installed in a disposal field treating the effluent from an individual household. The field contained 10 laterals, each 17.1 m long. Four test holes (one at each corner of the bed) were installed to monitor groundwater depths and water quality. Walker et al. (1973) installed ground water observation wells in four disposal field systems. The number of wells at each site ranged from 7 to 18. In each case the number of wells was sufficient to construct potentiometric maps in order to establish the direction of groundwater movement.

The authors of US EPA (1980) suggested that if several observation wells are strategically placed in the vicinity of disposal fields, local groundwater gradients can be established by measuring the water surface elevation in each well. They suggested the use of piezometers to establish whether artesian conditions exist. The monitoring of groundwater levels is used primarily as a predevelopment tool to indicate where the drainage networks should be placed.

5.3 MONITORING GROUNDWATER QUALITY

Numerous authors have discussed the monitoring of groundwater in order to study the effects of septic tank effluents and treatment efficiencies on the groundwater regime. They usually involve groundwater monitoring in the unsaturated zone below the disposal fields. Monitoring in the unsaturated zone usually provides an early warning of potential groundwater contamination.

The US EPA (1983) has described the use and operation of the pressure-vacuum lysimeter, the device most commonly used to sample groundwater in the unsaturated zone. A typical lysimeter has been illustrated in Figure 5.1. The authors stated that in an optimum arrangement, lysimeters are installed at various depths in the unsaturated zone. Bentonite plugs are placed at the top and bottom of each hole during backfilling to prevent channeling of contaminated surface water directly to the lysimeter. These same authors suggested that lysimeters can be installed horizontally into the soil or at angles along the edge of the site. The porous ceramic cup in each

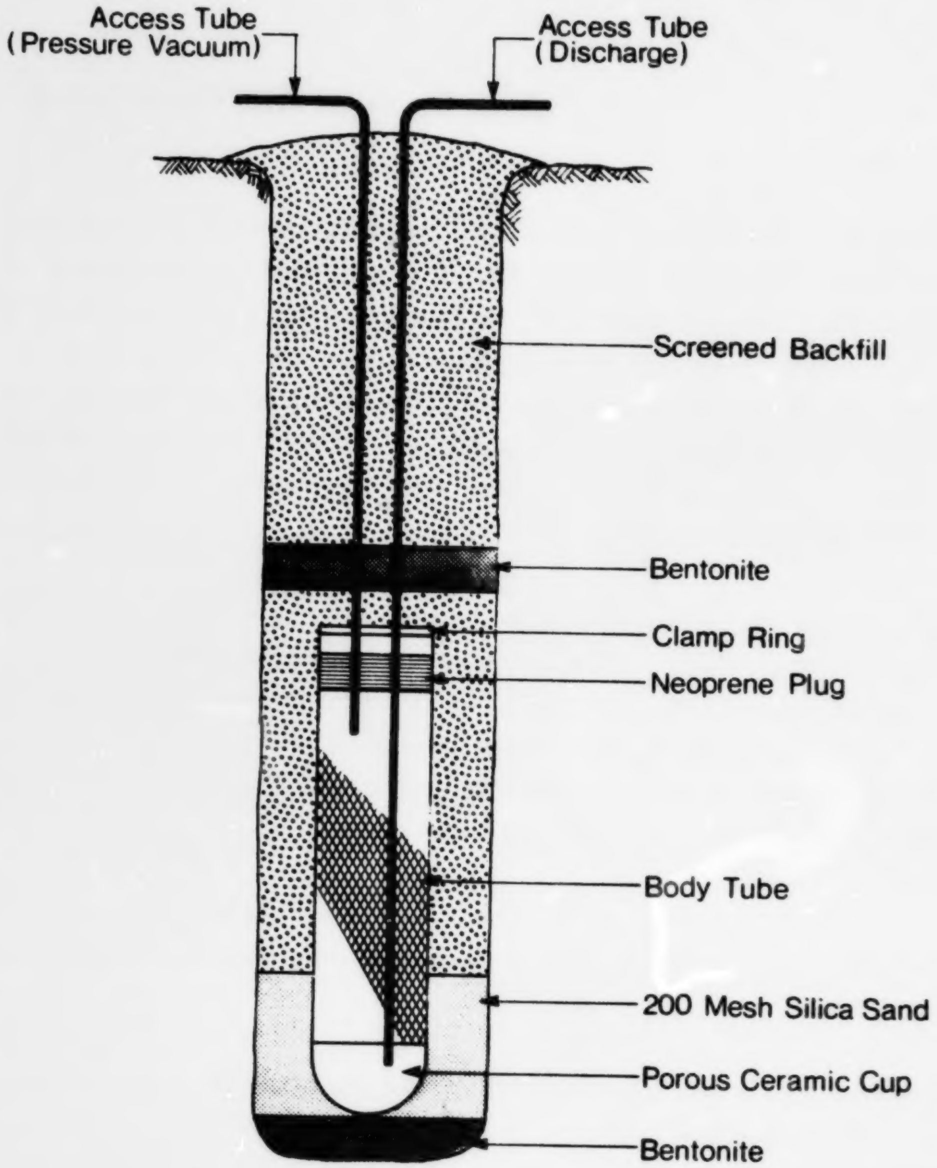

Access Tube (Pressure Vacuum)
Access Tube (Discharge)
Screened Backfill
Bentonite
Clamp Ring
Neoprene Plug
Body Tube
200 Mesh Silica Sand
Porous Ceramic Cup
Bentonite

Figure 5.1

TYPICAL
PRESSURE / VACUUM LYSIMETER
FOR LEACHATE MONITORING

lysimeter should be surrounded by a slurry of wet, fine quartz which ensures hydraulic continuity with the surrounding soil.

The same authors have described the monitoring of groundwater in the saturated zone with the use of wells as illustrated in Figure 5.2. They suggested that the number of wells and their depth placement depends on the location of the water table and the direction of groundwater flow. If several aquifers could be affected, a set of monitoring wells may be required for each aquifer. They stated that the depth of each well is dependent on the depth of the aquifer(s) being sampled and the predicted pathway of potential migrating contaminants. They suggested that consideration should be given to the following factors:

1) Soil and rock formations existing on the site.
2) Direction of groundwater flow and anticipated rate of movement.
3) Depth of seasonal high water table, and an indication of seasonal variations in groundwater depth and direction of movement.
4) Nature, extent, and consequences of groundwater mounding, which may occur above the naturally occurring water table.
5) Depth of impervious layers.

Generally, if monitoring is required, three or more monitoring wells are installed, as follows (US EPA, 1983):

1) One background well located upstream, and not affected or contaminated by percolating effluent.
2) One or more (depending on site size and hydrogeological factors) wells located off-site downgradient from the site, and used to detect migration of pollutants.
3) One or more on-site wells located in the zone of maximum pollutant concentration.

They strongly suggested the installation of monitoring wells during the site selection and/or design investigations to establish background ground water quality including any seasonal fluctuations before disposal field construction.

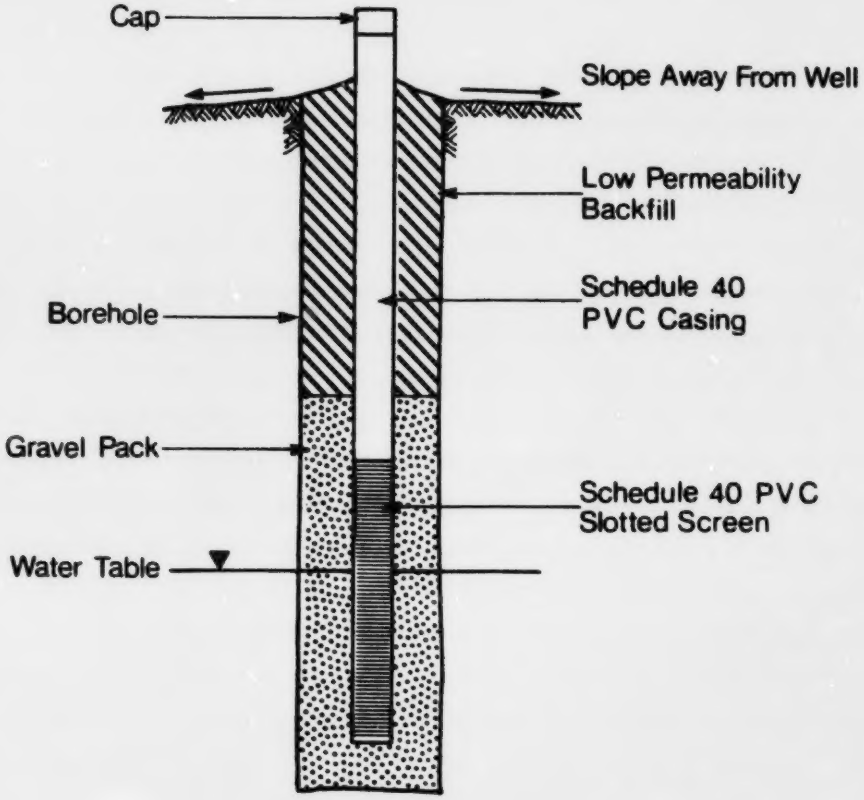

Cap
Slope Away From Well
Low Permeability Backfill
Schedule 40 PVC Casing
Borehole
Gravel Pack
Schedule 40 PVC Slotted Screen
Water Table

Figure 5.2

TYPICAL
MONITORING WELL SCREENED
OVER A SINGLE VERTICAL INTERVAL

Loehr et al. (1979) provided practical guidelines for groundwater sampling:

1) A measured amount of water equal to or greater than three times the amount of water in the well and/or gravel pack should be exhausted from the well before sample collection. In the case of very low-permeability soils, the well may have to be exhausted and allowed to refill before a sample is collected.

2) Pumping equipment should be thoroughly rinsed before use in each monitoring well. Water pumped from each monitoring well should be discharged to the ground surface away from the wells to avoid recycling of flow in high-permeability soil areas.

3) Samples should be collected, stored, and transported to the laboratory in a manner to avoid contamination or interference with subsequent analyses.

The frequency of sample collection is dependent upon the goals of a particular groundwater monitoring program (US EPA, 1983). The authors estimated that the rate of pollutant travel in a given hydrogeologic setting will indicate intervals of time which will show a change in water quality. They cautioned that arbitrary selection of sampling frequency may not reveal the true picture of groundwater quality at the disposal field site.

SECTION 6

REGULATORY REQUIREMENTS

6.1 THE EVALUATION OF REGULATORY DESIGN CODES

The manual of Septic Tank Practise, published by the U.S. Public Health Service in 1957, provided the first major compilation of septic tank - disposal field design codes based on sound research results (Kreissl, 1982a). Prior to that time, regulatory agencies in the United States administered a wide variety of design codes. The Manual of Septic Tank Practice had such an influence on state regulatory codes that by 1971, a survey showed that modal values for septic tank - disposal field systems were identical to the U.S. Public Health Service Manual (Patterson et al, 1971). During the past 10 years there has again been a divergence of regulation standards as further research studies have unveiled new site specific design and operating information. The Virginia Department of Health (1980) found through a survey of state regulations that all but three of the 47 responding states had recently revised their conventional septic tank - disposal field regulations.

To illustrate further the divergence in design codes which has occurred during the past ten years, the results of a study by Plews (1975) are reported in Tables 6.1 and 6.2. They illustrate a variability of criteria and demonstrate some of the following areas of concern:

1) the movement of pollutants from absorption fields as indicated by setback distances and minimum percolation restrictions;

2) the flow net below absorption trenches as indicated by minimum spacings between trenches of varying widths; and

3) aeration, freezing and evapotranspiration phenomena as indicated by minimum cover specifications.

TABLE 6.1

SEPTIC TANK DESIGN - STAGE REGULATORY CODES

(Plews, 1975; Machmeier, 1985)

States	Tank Size in Gallons Number of Bedrooms 1	2	3	4	5	Minimum Water Depth (Feet)	Open Discharge
Alabama	1000	1000	1000	1200	1400	4	
Alaska	750	750	900	1000	1250	4	No
Arizona	960	960	960	1200	1500	4	No
Colorado	750	750	900	1000	1250	No Minimum	Yes
Connecticut	1000	1000	1000	1250	1500	1.5	No
Florida	750	750	900	1000	1200	1.5	No
Georgia	750	750	900	1000	1250	No Minimum	No
Idaho	750	750	900	1000	1250	4	No
Indiana	750	750	900	1100	1250		No
Iowa	750	750	1000	1250	1500	1.5	Yes
Kentucky	750	750	900	1000	1250		No
Louisiana	500	750	900	1150	1400	None	Yes
Maine	750	750	900	1000	1250	2	Yes
Minnesota	750	750	1000	1000	1500	3	No
Montana	750	750	900	1000	1250	4	No
Nebraska	750	750	900	1000	1250		No
Nevada	1000	1000	1000	1000	1250	4	No
New Hampshire	750	750	900	1000	1250	4	No
North Carolina	750	750	900	1000	1250	2	No
Ohio	1000	1000	1500	2000	2000	4	Yes
Oregon	750	750	900	1000	1250	1.5	No
Pennsylvania	900	900	900	1000	1100	4	No
Rhode Island	750	750	900	1000	1250	3	No
South Dakota	1000	1000	1000	1250	1500	4	No
Tennessee	750	750	900	1000	1250	4	No
Utah	750	750	900	1000	1250	1	No
Virginia	30 100	Hour Detention and Gallons Per Bedroom Per Day				No Minimum	Yes
Washington	750	750	900	1000	1250	3	No
West Virginia	750	750	900	1000	1250	4	No
Wisconsin	750	750	975	1200	1375	3	No
Wyoming	750	750	900	1000	1250	3	Yes

TABLE 6.2

ABSORPTION FIELD DESIGN - STATE REGULATORY CODES

(Plews, 1975; Machmeier, 1985))

States	Setbacks		Minimum Spacing (Feet)	Minimum Cover (Inches)	Minimum Percolation Restrictions	Trench Widths (Inches)	Sizing
	Well	Surface Water					
Alabama	50-75		6	6	None	18-36	Perc
Alaska	50-100	50-100	6	12	None	12-36	Perc & Soils
Arizona	50-100	100	6	12	None	12-18	Perc
Colorado	100	50	6	12	None	18-36	Perc
Connecticut	75	50	6-9	6	None	18-36	Perc
Florida	75-100	50	6-8	12	None	18-24	Perc & Soils
Georgia	100	50	10	12	None	18-36	Perc & Soils
Idaho	100	100-300	6	12	None	12-36	Perc & Soils
Indiana	50-100	50	6-7.5	12	None	18-36	Perc
Iowa	100-200	25	7.5	12	None	18	Perc
Kentucky				None			Perc
Louisiana	100			6-12	None	12-18	Perc
Maine	100-300	50-100	10	2-6	None	24	Soils
Minnesota	50-100	50-150	None	6	Yes	18-36	Perc
Montana	100	100	6	12	Yes	12-36	Perc & Soils
Nebraska	100	50	6	6	No	18-36	Perc
Nevada	100	100	6	4-6	Yes	12-24	Perc
New Hampshire	75	75	6-7.5	6	None	12-36	Perc
Ohio	50		6	6	None	8-30	Soils
Oregon	50-100	50-100	10	6	None	24	Soils
Pennsylvania	100	50	6	12	Yes	12-36	Perc
Rhode Island	100	50	6	12	None	18	Perc
South Dakota	100	100	6		Yes		Perc
Tennessee	50	50	6	12	None	18-36	Perc & Soils
Utah	100	100	6-7.5	12	None	12-36	Perc
Virginia	35-100	50-100	6-9	None	None	18-36	Perc & Soils
Washington	75-100	100	6	6	Yes	18-36	Perc & Soils
West Virginia	100	100	6	12	None	12-36	Perc
Wisconsin	50-100	50	10	12	None	18-36	Perc & Soils
Wyoming	100	50	6-7.5	6-12	None	12-36	Perc

The variability is due partially to climatic conditions but even more importantly, to an absence of reliable methods to evaluate site specific design conditions.

6.2 CURRENT REGULATORY CODES

In an effort to compare some of the current regulatory requirements for the design and installation of septic tank - disposal field systems, local codes of practice were obtained for a number of states and provinces in North America. A detailed summary of these regulatory codes is presented in Tables 6.3 to 6.8.

The sizing of disposal fields is one of the most important design parameters for septic tank - disposal field systems, particularly when they are used for large sewage flows. Disposal costs and constraints are usually dictated by the availability of land for disposal fields. The tables illustrate a wide variability in the basis for sizing of beds for effluent disposal. Observations on different sizing practices are a follows:

1) Some jurisdictions place upper and lower limits on the allowable percolation rates for the use of disposal beds. In most cases a lower limit of 60 min/25 mm has been used; the provinces of Alberta and Manitoba allow the use of beds and trenches in soils with percolation rates lower than 90 min/25 mm.

2) Canadian practice has consisted of defining trench width and sidewall depth within narrow ranges and then sizing field on the basis of total trench length; U.S. practice appears to favour variable trench widths and depths and then sizing trenches on the basis of total bottom area.

3) There is considerable variation between states and provinces on bed sizing within soil percolation ranges and soil textural classes.

4) Some jurisdications use the percolation test almost exclusively to size disposal fields, others use a combination of percolation data and soil textural analysis; the State of Oregon utilizes only soil textural analysis.

TABLE 6.3

SEPTIC TANK SETBACK REQUIREMENTS

Regulatory Code or Agency	Building	Property Boundary	Septic Tank Setback Requirements (m)						
			Wells	Surface Water	Cut or Embankment	Swimming Pool	Water Service Pipes	Walks and Drives	Large Trees
Alberta	1	1	10	10	ND[3]	ND	ND	ND	ND
Saskatchewan	0.9	ND	ND	ND	ND	ND	ND	ND	ND
Manitoba	1	3	8	15	8	3	ND	2.0	ND
Ontaio	1.5	3	15	15	ND	15	ND	ND	ND
Nova Soctia	ND	30 max.	ND	ND	ND	ND	ND	ND	ND
CMHC[1]	1.5	3	15	15	7.5	3	3	1.5	3
Minnesota	3	3	15	15-46	ND	ND	3	ND	3
Wyoming	1.5	3	15	15	ND	7.5-15	ND	ND	ND
Wisconsin	ND	3	ND	15	ND	ND	3	ND	ND
Colorado	1.5	3	15	15	3	ND	3	ND	ND
Oregon	1.5	3	15	15	3-8	ND	3	ND	ND
Ten State Standards[2]	3	3	15	15	ND	ND	3	ND	ND

[1] Central Mortage and Housing Corporation, Canada

[2] Great Lakes - Upper Mississippi River Board of State Sanitary Engineering, 1980 Edition

[3] Not Specifically Defined

TABLE 6.4

SEPTIC TANK DESIGN CRITERIA

Regulatory Code or Agency	Working Capacity (L) Number of Bedrooms 1	2	3	4	5	6	Minimum Working Capacity (L)	Minimum Water Depth (m)	Minimum Number of Compartments	Inlet and Outlet Baffles
Alberta	1800	1800	2045	2700	3400	3400	1800	1.2	ND[3]	ND
Saskatchewan	1800	1800	1800	2250	2700	3150	1800	1.2	ND	Yes
Manitoba	1800	1800	2270	2270	-	-	1800		2	
Ontario	2270	2270	2720	3400	4100	4900	2270	1.2	ND	Yes
Nova Scotia	2700 to 4500	2700 to 4500	2700 to 4500	3250 to 4500	4100 to 4500	ND	2700 to 4500	1.2	ND	Yes
CMHC[1]	2300	2300	2700	3400	4100	5000	2300	1.2	2	Yes
Minnesota	2800	2800	3800	4700	5700	5700	2800	0.8	1	Yes
Wyoming	3800	3800	3800	5100	6400	7700	3800	.8	1	Yes
Wisconsin	2800	2800	3700	4500	5400	6250	2800	.9	1	Yes
Colorado	2800	2800	3400	3800	4750	5700	2800	.8	2	Yes
Oregon	3800	3800	3800	3800	4700	4700	3800	1.2	ND	Yes
Ten State Standards[2]	2800	2800	3800	4700	5700	6700	2800	1.2	ND	Yes

[1] Central Mortgage and Housing Corporation, Canada

[2] Great Lakes - Upper Mississippi River Board of State Sanitary Engineers, 1980 Edition

[3] Not Specifically Defined

TABLE 6.5

DISPOSAL FIELD SETBACK REQUIREMENTS

			Disposal Field Setback Requirements (m)						
Regulatory Code or Agency	Building	Property Boundary	Wells	Surface Water	Cut or Embankment	Swimming Pool	Water Service Pipes	Walks and Drives	Large Trees
Alberta	10	1.5	15	15	ND[3]	ND	ND	ND	ND
Saskatchewan	11	3	15	30	15	8	8	2	ND
Manitoba	11	8	15	30	15	8	8	2ND	
Ontario	3-8	3	15-30	15	ND	15	ND	ND	ND
Nova Scotia	5	3-15	15-50	30-60	ND	ND	ND	ND	ND
CMHC[1]	7.5	3	30	15	15	7.5	7.5	1.5	3
Minnesota	6	3	15-30	15-45	ND	ND	ND	ND	ND
Wyoming	3	3	30	15	ND	ND	7.5-30	ND	ND
Wisconsin	7.5	1.5	15	15	ND	4.5	3	ND	ND
Colorado	6	7.5	30	15	7.5	ND	15	ND	ND
Oregon	3	3	30	30	8-16	ND	3	ND	ND
Ten State Standards[2]	3	3	15	15	ND	ND	ND	ND	ND

[1] Central Mortage and Housing Corporation, Canada

[2] Great Lakes - Upper Mississippi River Board of State Sanitary Engineering, 1980 Edition

[3] Not Specifically Defined

TABLE 6.6

DISPOSAL FIELD SITE SUITABILITY CRITERIA

	Minimum Depth to Groundwater (cm)	Maximum Site Slope	Minimum Percolation Rate (min/25 mm)	Maximum Percolation Rate (min/25 mm)	Minimum Depth to Impervious Layer (cm)	Maximum Length of Trenches (m)
Alberta	ND[3]	ND	ND	ND	ND	ND
Saskatchewan	150-750	ND	1	60	ND	18.3
Manitoba	ND	ND	ND	ND	ND	18.0
Ontario	90	25%	1	60	90	18-30
Nova Scotia	ND	ND	ND	ND	ND	ND
CMHC[1]	90	20%	5	60	90	18-30
Minnesota	90	ND	0.1	120	90	30
Wyoming	120	6%	ND	60	120	30
Wisconsin	90	20%	ND	60	90	ND
Colorado	120	30%	5	60	120	30
Oregon	60-120	30%	ND	ND	75	38
Ten State Standards[2]	90	ND	ND	60	90	30.5

[1] Central Mortage and Housing Corporation, Canada

[2] Great Lakes - Upper Mississippi River Board of State Sanitary Engineering, 1980 Edition

[3] Not Specifically Defined

TABLE 6.7

DISPOSAL FIELD DESIGN CRITERIA

Regulatory Code or Agency	Basis for Bed Sizing	Number of Percolation Tests	Depth of Trenches (cm)	Width of Trenches (cm)	Depth of Backfill (cm)	Depth of Gravel (cm)	Distance Between Laterals	Total Length of Trenches Per Bedroom (m) Percolation Rate (min/25 mm)					
								0-20	10-20	20-30	30-60	60-90	Over 90
Alberta	Perc	2 per Site	60-75	45	ND[3]	30	1.5	30	50	60	100	120	150
Saskatchewan	Perc	ND[3]	61	40	30	30	1.8	See Table 4.6					
Manitoba	Soil Type Perc	6 (where applicable)	61-91	41	30	30-60	2.0	See Table 4.6					
Ontario	Perc	ND	63-91	45	30	33	1.8	20	26	33	38-53	Not Allowed	
Nova Scotia	Perc	ND	70-85		30-45	40	1.2	Not Available					
CMHC[1]	Perc Soil Type	3 per Site	60-90	45	30-60	20-35	1.8	34	44	50	67	Not Allowed	
Minnesota	Perc	2 per Site	60-90	20-91	15-86	30-76	ND	See Table 4.6					
Wyoming	Perc	6	45-90	30-90	30	15-60	1.8	See Table 4.6					
Wisconsin	Perc	3	ND	30	ND	ND	1.8	See Table 4.6					
Colorado	Perc	3	ND	ND	25	30	1.8	See Table 4.6					
Oregon	Soil	ND	45-72	60	30	30	2.4	See Table 4.6					
Ten State Standards[2]	Perc	ND	45-90	45-90	ND	ND	1.5	See Table 4.6					

[1] Central Mortage and Housing Corporation, Canada

[2] Great Lakes - Upper Mississippi River Board of State Sanitary Engineering, 1980 Edition

[3] Not Specifically Defined

TABLE 6.8
BASIS FOR SIZING DISPOSAL FIELD TRENCH LENGTHS

	Comments
Alberta	For multi-unit developments use formula $m = \frac{LxP}{133}$ in which m = length in metres L = litres per day expected volume P = percolation rate in minutes per 25 mm
Saskatchewan	Trench length varies from 3 to 28 metres per person for percolation rates of 1 to 60 minutes per 25 mm. Proponent may use second option based on soil type which specifies minimum lengths for various soil types.
Manitoba	Minimum trench length varies from 55 m for 2,3 and 4 bedroom homes with sandy gravel soils to 76 m for 2 bedrooms, 82 metres for 3 bedroom and 91 metres for 4 bedrooms, where clay soil is used. Sizing of septic tanks for multi-unit developments is based on total daily sewage flow multiplied by 1.4; sizing of trenches is based on percolation rate and is defined in a table with capacity up to 13,640 l/day of sewage flow and percolation rates of 0-30, 30-60, 60-90 and greater than 90 minutes per 25 mm.
Ontario	Length of trenches varies from 45 m for 2 bedrooms and a soil percolation rate of 1 to 5 minutes to 105 m for a soil percolation rate of 50 to 60 minutes per 25 mm.
CMHC	Trench lengths can be selected either on the basis of measured percolation rate or by assigning a percolation rate to soil textural descriptions.
Minnesota	Sizing of trenches is based on bottom area conditions using measured percolation rates. Estimates of per capita total daily sewage flows vary even with the number of bedrooms and are a function of the type of home; they use the rationale that a luxury home will generate more sewage than a modest home.
Oregon	Sizing of trenches is based on trench bottom area according to soil texture and effective soil depth.
Colorado	The trench bottom absorption area in square feet is based on the formula: $A = \frac{Q}{5} t$ which Q = sewage flow in gallons per day and t is the percolation rate in minutes per inch.
Wyoming	Sizing of trenches is based on trench - bottom area requirements ranging from 6 square metres per bedroom for a percolation rate of 0 - 10 minutes per 25 mm to 31 square metres per bedrom for a rate of 60 minutes per 25 mm.
Ten State Standards	Sizing of trenches is based on trench bottom area requirements ranging from 15 square metres per bedroom for a percolation rate of 0 - 10 minutes per 25 mm to 31 square metres per bedroom for a rate of 60 minutes per 25 mm.

These all illustrate that there is an obvious lack of consensus on the methods to be used and the actual values to be selected for the sizing of disposal fields.

The inaccuracies associated with the percolation test and the absence of any one definitive test for measuring the soils' capability to remove and to renovate wastewater were referenced earlier. Since the proper sizing of disposal fields plays such a vital role in their successful operation and in the protection of the surrounding environment, there is an obvious trend towards adoption of conservative design criteria. There is a general uncentainty about the effectiveness of sidewall areas on soil absorption capabilities and constraints. Additional factors which may account for the large variability in sizing between jurisdictions may be related to variability in soil types, particularly soil parent materials, regional water quality concerns and finally a desire on the part of regulatory agencies to discourage the proliferation of septic tank - disposal field systems within the rural-urban fringe.

There is also wide variation in the sizing of septic tanks for single-family dwellings. Minimum sizes range from 1800 litres in the Prairie Provinces to 3800 litres in Oregon and Wyoming. The larger values probably reflect an attempt to adopt conservative sizes to ensure sufficient liquid working volume even where homeowners neglect to remove sludge and scum every two or three years. The smaller sizing for the Prairie provinces serves to minimize the potential for freezing in the septic tank and the disposal field since it shortens the retention time in the system. The use of larger tanks in more moderate climates provides maximum opportunity for the settling of the suspended solids which are instrumental in clogging disposal fields.

In an evaluation of trends in septic tank - disposal field design codes for the United States during the years 1947 to 1980, Kreissl (1982a) made the following observations:

1) Current code - makers feel that larger septic tanks are advantageous but they are becoming less restrictive on the shape of the tank. Earlier intent to improve tank performance through better internal design has apparently given way to perceived benefits of larger tanks. (Kreissl attributed this trend in sizing to reduced pumping frequency, and the concerns for both

increased wastewater flow due to modern fixtures and the widespread use of garbage grinders).

2) There are extreme variabilities in the requirements for utilizing the percolation test and in the test methods suggested. Despite its shortcomings the percolation test continues to be extremely popular, probably because of the complexity of alternative tests and a lack of information on how to use the more accurate data generated by them. There is, however, a trend away from the 'blind' use of percolation tests and a tendency to rely on additional site evaluation data including soil textural analysis, soil profiles, landscape location and drainage, and prior experience with disposal fields in the identified soil.

3) There is a trend toward consistency in minimum trench width (30 cm) and depth (60 cm), a reduction in separation distances between trenches from 2.1 m to 1.8 m, and a reduction in minimum fill requirements. He attributed these trends to changes in equipment employed for digging trenches, a diminished concern for freezing and mechanical damage, and an attempt to use evapotranspiration assistance more effectively.

4) The required sizing of conventional soil disposal fields has generally increased over the intervening period, probably due to unsatisfactory performance of older, smaller systems. He noted that this conservative trend ignores the negative economics of using larger beds and increases the potential for groundwater contamination by increasing disposal field longevity through bed enlargement without improving distribution systems.

5) Horizontal setback requirements to water supplies were quite varied but had generally increased from 15 m to 30 m in the case of wells and had remained constant at 15 m for surface waters. He attributed the increased setback requirement for water wells to a concern for pathogen transport but pointed out that the mere codification of a minimum setback distance in and of itself does not ensure against pathogenic contamination, nor does it prevent the passage of nitrates to wells and surface water.

6) There is a trend toward a requirement to provide a disposal field replacement area equivalent to 100 percent of the original system area. He felt that this trend admits failure to present design or control criteria and a lack of understanding of system functioning.

It is readily apparent that the criteria for design of septic tank - disposal field systems and the codes which enforce them require further research to develop more refined, cost effective and environmentally safe wastewater disposal systems.

SECTION 7

FUTURE TRENDS

Sewage treatment practices during the past 20 to 30 years have seen the trend towards large collective community treatment systems providing full secondary treatment and in some cases tertiary treatment. These systems by necessity usually employ energy and labour intensive aeration processes, and in most cases discharge their renovated effluents to surface receiving waters. With recent escalations of energy costs and concerns for surface water contamination, there has been a renewed interest in exploring cheaper alternatives, most of which involve using some form of disposal to land. The use of septic tank - disposal field systems will become increasingly more attractive for small communities where the land resources, particularly for disposal fields, are available. There is growing pressure, therefore, to refine current methods of septic tank and disposal field sizing and to develop new techniques which will reduce the need to use conservatively safe codes.

Kreissl (1982b), reporting current research activities and needs in the United States, discussed the following:

1) development of alternative pretreatment processes for the use of soil disposal fields;

2) modifications to the design of septic tanks;

3) the development of pressure distribution systems for effluent feeding to disposal fields;

4) the refinement of alternative bed systems;

5) the development of better disposal field designs for problem soils including refinement of mound systems, evapotranspiration beds for warmer climate and submerged sand filters;

6) the development of wastewater separation and recycling techniques; and

7) the development of waterless toilet designs including incinerating toilets, reduced flush toilets, and various types of chemical and biological composting systems.

In his discussion of development needs for alternative pretreatment processes, he noted the failure of current aerobic systems to demonstrate consistent performance. Therefore, any attempt to reduce the size of soil absorption fields for aerobic systems would be short-sighted, at best, without future refinements; their substitution for septic tanks at present cannot be justified on economic grounds. The use of anaerobic upflow filters, as a means of improving septic tank performance is potentially attractive because the idealized hydraulics of these devices allow improvement in effluent quality. Design improvements to the septic tank itself which are gaining increasing acceptance with code-makers include the use of multiple rather than single compartment tanks, and improved outlet arrangements.

The author noted that significant progress is being made in the research and development of improved effluent distribution and disposal field designs and viable alternatives to overcome specific site limitations. The uniform distribution of effluent to disposal trenches with the use of a pressure distribution system is gaining increasing favour and the concept has been demonstrated not only for conventionally acceptable sites but also where high groundwater and coarse soils exist. The idea of restoring the original permeability of soil disposal beds by allowing them to rest with the use of alternating bed systems is gaining acceptance. Current research efforts include defining the required length of resting periods and identifying the independent variables contributing to permeability restoration in different soils.

One modification to the original NODAK mound system which has met a great deal of success in Wisconsin employs pressure distribution and sand as a fill material (Harkin et al., 1979). Other less successful variations include the use of plastic liners and clay diking and oversized mounds employing gravelly loam fill. As explained previously, mounds are suitable alternatives for soils with very low infiltration capacity. Other recently developed alternatives for problem soils which have potential for more widespread application in the future (Kreissl, 1982b) include artificial drains, shallow trench systems, evapotranspiration systems and buried sand filters. The latter two have very limited potential for Alberta because of severe climatic constraints.

Several authors have discussed the development and need for community management systems for multi-unit septic tank - disposal field systems (Wilson et al., 1979). Considerable research, particularly in the United States, has been devoted towards the definition of management alternatives already in use and the development of user guides to assist local officials and engineers to choose optimum management strategies which are in compliance with the local administrative and legal constraints and with the alternative technologies which might be used in their area. The need for implementation of good management systems for not only the design but also the operation of large septic tank - disposal field systems is as important to the success of their operation as the development of sound design criteria.

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